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MARCH

1932

VOLUME 58 NUMBER 3

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PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 58 MARCH, 1932 No. 3

UNBOUND

TECHNICAL PAPERS
DISCUSSIONS
APPLICATIONS FOR ADMISSION AND TRANSFER

Published monthly, except June and July, at 99–129 North Broadway, Albany, N. Y., by the American Society of Civil Engineers, Editorial and General Offices at 33 West Thirty-ninth Street, New York, N. Y. Reprints from this publication may be made on condition that the full title of Paper, name of Author, page reference, and date of publication by the Society, are given.

Entered as Second-Class Matter, December 28, 1931, at the Post Office at Albany, N. Y., under the Act of March 3, 1879. Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

Subscription (if entered before January 1) $8.00 per annum. Price $1.00 per copy.

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CURRENT PAPERS AND DISCUSSIONS

		Discussion closes
Analysis of Continuous Frames by Distributing Fixed-End Moments. *Hardy Cross*	May, 1930	
Discussion...Sept., Oct., Nov., 1930, Jan., Feb., Mar., May, Sept., Nov., 1931, Jan., Mar., 1932		Closed
Highway Location: General Considerations. *Gibb Gilchrist*	Nov., 1930	
Discussion (Author's closure)	Feb., Oct., 1931, Mar., 1932	Closed
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Discussion	Feb., May, 1931	Closed
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Discussion	May, 1931	Closed
Surveys for Swift River Reservoir of the Boston Metropolitan Water Supply. *N. LeRoy Hammond*	Feb., 1931	
Discussion (Author's closure)	May, Nov., 1931, Mar., 1932	Closed
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Discussion (Author's closure)	Sept., Nov., 1931, Mar., 1932	Closed
Gravity Dams Arched Down Stream. *B. J. Lambert*	Mar., 1931	
Discussion (Author's closure)	Aug., Oct., 1931, Mar., 1932	Closed
An Analysis of Multiple-Skew Arches on Elastic Piers. *J. Charles Rathbun*	Apr., 1931	
Discussion (Author's closure)	Sept., Nov., 1931, Jan., Mar., 1932	Closed
Run-Off—Rational Run-off Formulas. *R. L. Gregory and C. E. Arnold*	Apr., 1931	
Discussion	Sept., Nov., Dec., 1931, Jan., Feb., 1932	Uncertain
Manufacturing Concrete of Uniform Quality. *William M. Hall*	May, 1931	
Discussion	Sept., Nov., Dec., 1931, Mar., 1932	Closed
Financing Street and Highway Improvements. *R. W. Crum*	Aug., 1931	
Discussion	Oct., 1931, Feb., 1932	Mar., 1932
Design of Large Pipe Lines. *Herman Schorer*	Sept., 1931	
Discussion	Nov., Dec., 1931, Jan., Feb., 1932	Mar., 1932
Highway Construction Management. *T. Warren Allen*	Sept., 1931	
Discussion	Nov., Dec., 1931	Mar., 1932
Construction Management. *Richard Hopkins*	Sept., 1931	
Discussion	Jan., Feb., 1932	Mar., 1932
Soil Mechanics Research. *Glennon Gilboy*	Oct., 1931	
Discussion	Dec., 1931, Jan., Feb., 1932	Apr., 1932
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Discussion	Feb., Mar., 1932	Apr., 1932
Construction Work on a Federal Reclamation Project. *Morris Mason*	Oct., 1931	
Discussion	Dec., 1931, Jan., 1932	Apr., 1932
Water Pressures on Dams During Earthquakes. *H. M. Westergaard*	Nov., 1931	
Discussion	Feb., 1932	May, 1932
Western Highway Practice with Special Reference to California. *C. S. Pope*	Nov., 1931	
Discussion	Jan., Mar., 1932	May, 1932
Low-Cost Bituminous Roads. *Walter N. Frickstad*	Nov., 1931	
Discussion	Feb., Mar., 1932	May, 1932
Effect of Bending Wire Rope. *F. C. Carstarphen*	Dec., 1931	May, 1932
Lake Champlain Bridge. *Charles M. Spofford*	Dec., 1931	
Discussion	Mar., 1932	May, 1932
Construction Plant and Methods for Erecting Steel Bridges. *A. F. Reichmann*	Dec., 1931	
Discussion	Mar., 1932	May, 1932
Fulton Street, East River Tunnels, New York, N. Y. *Miles I. Killmer*	Dec., 1931	May, 1932
Wind-Bracing Connection Efficiency. *U. T. Berg*	Jan., 1932	May, 1932
Public Supervision of Dams: A Symposium	Jan., 1932	
Discussion	Mar., 1932	May, 1932
Stereo-Topographic Mapping. *C. H. Birdseye*	Jan., 1932	May, 1932
Determination of Principal Stresses in Buttresses and Gravity Dams. *W. H. Holmes*	Jan., 1932	May, 1932
Economic Proportions and Weights of Modern Highway Cantilever Bridges. *J. A. L. Waddell*	Feb., 1932	May, 1932
Design Characteristics of the Reading Overbuild Transmission Line. *Frederick W. Deck*	Feb., 1932	May, 1932
Stresses in Reinforced Concrete Due to Volume Changes. *C. P. Vetter*	Feb., 1932	May, 1932
Wind-Bracing in Steel Buildings: Second Progress Report of Sub-Committee No. 31, Committee on Steel, of the Structural Division	Feb., 1932	May, 1932

CONTENTS FOR MARCH, 1932

PAPERS

PAGE

Suspension Bridges Under the Action of Lateral Forces.
By Leon S. Moisseiff, M. Am. Soc. C. E., and Frederick Lienhard, Esq 301

Construction Methods on the Welland Ship Canal.
By E. G. Cameron, Esq 317

Stresses in Inclined Arches of Multiple-Arch Dams.
By George E. Goodall and Ivan M. Nelidov, Associate Members, Am. Soc. C. E. ... 387

DISCUSSIONS

Analysis of Continuous Frames by Distributing Fixed-End Moments
By Messrs. A. W. Earl, A. Floris, I. M. Nelidov, E. A. MacLean, George M. Dillingham and Donald E. Larson 409

Highway Location: General Considerations.
By Gibb Gilchrist, M. Am. Soc. C. E 431

Surveys for Swift River Reservoir of the Boston Metropolitan Water Supply.
By N. LeRoy Hammond, M. Am. Soc. C. E 434

Flow of Ground-Water as Applied to Drainage Wells.
By M. R. Lewis, M. Am. Soc. C. E 436

Gravity Dams Arched Down Stream.
By Messrs. Floyd A. Nagler and B. J. Lambert 438

An Analysis of Multiple-Skew Arches on Elastic Piers.
By Messrs. Richard M. Hodges, and J. Charles Rathbun 440

Manufacturing Concrete of Uniform Quality.
By John H. Griffith, M. Am. Soc. C. E 450

Engineering Features of the Illinois Waterway.
By Messrs. David A. Molitor, and Charles A. D. Young 453

Western Highway Practice, with Special Reference to California.
By Messrs. H. S. Kerr, E. Q. Sullivan, and T. A. Corry 464

CONTENTS FOR MARCH, 1932 (*Continued*)

PAGE

Low-Cost Bituminous Roads.
By Messrs. A. H. Hinkle, Claude P. Owens, Carlton N. Conner, Harold L. Tilton, and L. I. Hewes 470

Lake Champlain Bridge.
By Messrs. Robert W. Abbett, Henry W. Troelsch, Jacob Feld, and Clarence W. Hudson. 493

Construction Plant and Methods of Erecting Steel Bridges.
By Messrs. Howard P. Treadway, Benjamin W. Guppy, R. A. Van Ness, and J. G. Tripp 498

Public Supervision of Dams: A Symposium.
By H. de B. Parsons, M. Am. Soc. C. E. 502

For Index to all Papers, the discussion of which is current in PROCEEDINGS, *see page 2*

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SOCIETY AFFAIRS

Annual Report of the Board of Direction for the Year Ending December 31, 1931 . . . . . following page 504

MEMBERSHIP

Applications for Admission and Transfer ..... following page 26 of Society Affairs

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

SUSPENSION BRIDGES UNDER THE ACTION OF LATERAL FORCES

BY LEON S. MOISSEIFF,[1] M. AM. SOC. C. E., AND FREDERICK LIENHARD,[2] ESQ.

SYNOPSIS

Lateral forces, such as horizontal wind pressures, when acting on a suspension bridge are sustained by the cables and the stiffening trusses, which transmit the resulting reactions to the towers and abutments. The hangers which connect the trusses to the cables force the two loaded systems to work together so that the deformation on one system exerts its influence on the other.

As soon as the bridge swings out of its vertical plane the effect of the dead and live load comes into action and tends to reduce the horizontal deflections. Consequently, a redistribution of the acting wind load follows until equilibrium is established between all acting forces.

Compared with the suspended structure, the cables themselves offer a small exposed area to wind pressure, but part of the forces which act on the suspended structure are transmitted through the hangers to the cables. The magnitude of the transmitted forces depends on the respective stiffness of the two systems and the ratio of the wind pressure acting on them.

The stiffness of the cables is in direct, linear proportion to the pull in the cable and in inverse proportion to the square of the span length. The cable pull expresses the resisting effect of the gravity forces.

The stiffness of the truss is in direct, linear proportion to its horizontal moment of inertia and in inverse proportion to the fourth power of the span length.

The two methods treated in this paper will lead to the determination of the stresses in the stiffening trusses caused by wind pressure. As the title suggests, the first method —"The Uniform Distribution Method"— assumes a

NOTE.—Written discussion on this paper will be closed in *August, 1932 Proceedings.*

[1] Cons. Engr., New York, N. Y.

[2] Asst. Engr., The Port of New York Authority, New York, N. Y.

uniform distribution of wind pressure on the cables and trusses. It is an approximate procedure and the derived results, which, by assumption, can be expressed in explicit form, have to be judged accordingly. The second—"The Elastic Distribution Method"—does not assume the simplification of the first method. The results are derived from balancing the deflections of the two systems, the cables, and the trusses. Such distribution of wind pressure between cables and trusses must be found by trial computations which will result in a balanced position of the two combined systems.

The Uniform Distribution Method

This method applies only to a bridge the elevation of which is symmetrical about mid-span, and for the assumption that the wind exerts a pressure which is distributed uniformly over the entire span length. Both assumptions usually agree with practical cases.

To simplify the procedure further, the assumption is introduced that the

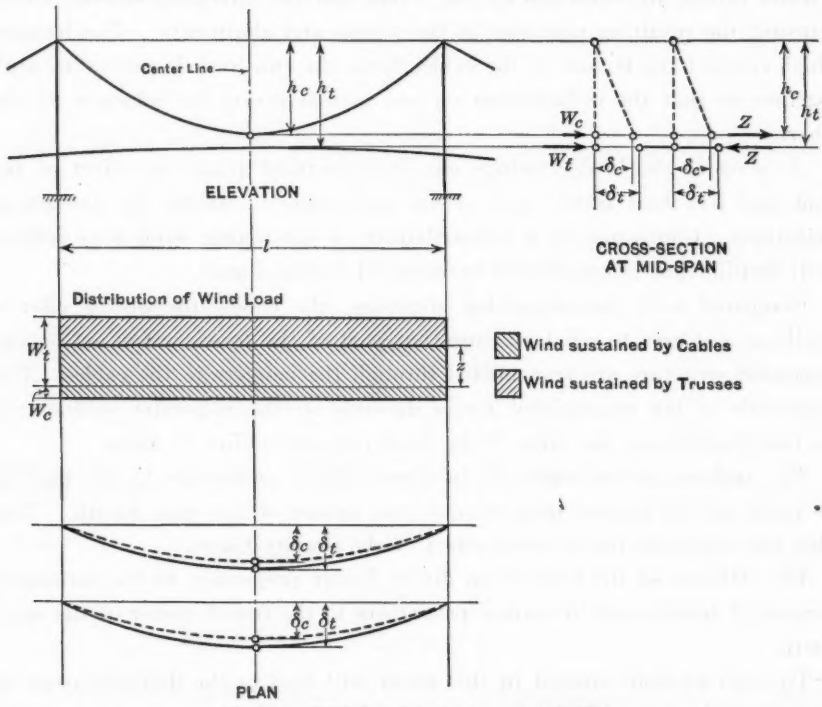

FIG. 1.—UNIFORM DISTRIBUTION METHOD.

wind pressure transferred from the trusses to the cables is also distributed uniformly over the entire length of the span. Consequently, the horizontal deformations of a cable will form a parabola and, since the elevation is a

curve following the same law, all points of the deflected cable will remain in one plane. This plane rotates about a line connecting the two bearing points of the cable on the tower saddles. To combine the deflections of the two systems, the trusses, and the cables, the assumption is made that at mid-span the deflected truss lies in the plane of the cable, as shown in cross-section in Fig. 1.

It can scarcely be expected that the actual behavior of the structure will conform to such simplifications. However, it affords a means of expressing in a simple form the effect of forces in two systems which depend on each other and yet behave so differently elastically. It will be shown that the method is of great usefulness, especially in cases where its results do not develop decided discrepancies with those of a more trenchant analysis, which is not based on such simplifying assumptions, but involves a much greater amount of work.

In the following analysis let:

l = length of span, in feet.
h_c = versine of cables, in feet.
h_t = versine of cables, in feet, plus length of hanger at mid-span, in feet.
δ_c = horizontal deflection of cables, in feet, at mid-span.
δ_t = horizontal deflection of trusses, in feet, at mid-span.
w_c = horizontal wind pressure on the cables, in pounds per foot of bridge.
w_t = horizontal wind pressure on the suspended structure, in pounds per foot of bridge; w_t also includes wind pressure on live load.
z = transferred wind from trusses to cables, in pounds per foot of bridge.
H = horizontal cable pull, in pounds, due to vertical load.
I = equivalent moment of inertia of trusses, in in.^{2} ft.^{2}
E = modulus of elasticity of steel, 29 000 000 lb. per sq. in.

The first assumption that the wind load is uniformly transferred from the trusses to the cables permits expressing the horizontal deflections of the two systems. For mid-span, it is:

In the cables,

$$\delta_c = \frac{(w_c + z)\, l^2}{8\, H} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and in the trusses,

$$\delta_t = \frac{5\, l^4}{384\, E\, I}\,(w_t - z) \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The assumption shown in cross-section in Fig. 1 gives the relation:

$$\frac{h_c}{h_t} = \frac{\delta_c}{\delta_t} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Combining Equations (1), (2), and (3), the transferred wind, z, is found as follows:

$$z = \frac{h_c\, l^2\, H\, w_t - 9.6\, E\, I\, h_t\, w_c}{h_c\, l^2\, H + 9.6\, E\, I\, h_t} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

In Equation (4) let $a = h_c\ l^2\ H$, and $b = 9.6\ h_t\ E\ I$; then,

$$z = \frac{a\ w_t - b\ w_c}{a + b} = \frac{w_t}{1 + \frac{b}{a}} - \frac{w_c}{1 + \frac{a}{b}} \cdot \quad \ldots\ldots\ldots\ldots (5)$$

It will be noted that $\frac{b}{a}$ represents the ratio of the stiffness of truss to that of the loaded cable.

Having determined the wind load transferred from the trusses to the cables and having assumed beforehand that it is uniform, the computation of the stresses in the trusses caused by it becomes simple.

The Elastic Distribution Method

This method derives the distribution of the wind pressure between the cables and the trusses by balancing their simultaneous deformations. The procedure is similar to the one used in the first method, namely, to find an expression for the deflections of the two systems and a relation between them.

Let h_x = hanger length, in feet.
δ_{cx} = horizontal deflection of cables, in feet.
δ_{tx} = horizontal deflection of trusses, in feet.
$\Delta\delta_x$ = difference in horizontal deflections between cables and trusses, in feet.
p_{cx} = wind sustained by cables, in pounds per foot of bridge.
p_{tx} = wind sustained by trusses, in pounds per foot of bridge.
p_{sx} = dead load of suspended structure and live load, in pounds per foot of bridge.
I_x = moment of inertia, in in.^{2} ft.^{2} The index, x, indicates the value at section x ft. from the end of the span.

Horizontal Deflections of the Trusses.—The horizontal deflection of the trusses, δ_{tx}, is given in the form of the differential equation of the elastic line:

$$d^2\delta_{cx} = \frac{M_x\ I_c}{E\ I_x}\ dx^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

which may be written,

$$E\ I\ \frac{d^2\delta_{tx}}{dx^2} = \frac{I_c}{I_x}\ M_x \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which, M_x is the bending moment in the trusses and I_c, any constant moment of inertia. The moment of inertia, I_c, is mostly so chosen that the term, $\frac{I_c}{I_x}$, becomes unity for as many sections as possible.

The algebraical integration of Equation (7) can only be solved for simple cases, that is, it must be possible to express the two terms, M_x and $\frac{I_c}{I_x}$, as functions of x. Such uniformity can scarcely be expected and it is best, therefore, to apply a well-known interpretation. It consists in recognizing that the right side of Equation (7), that is, the term, $\frac{I_c}{I_x}\ M_x$, where taken

as a loading on the trusses results in a moment diagram which, divided by $E\,I_c$, will give the desired deflection curve. The procedure is plainly shown in Fig. 2 (*a*) to Fig. 2 (*e*), inclusive.

These curves also show the procedure to determine the horizontal deflection of the trusses under the action of any distribution of horizontal forces.

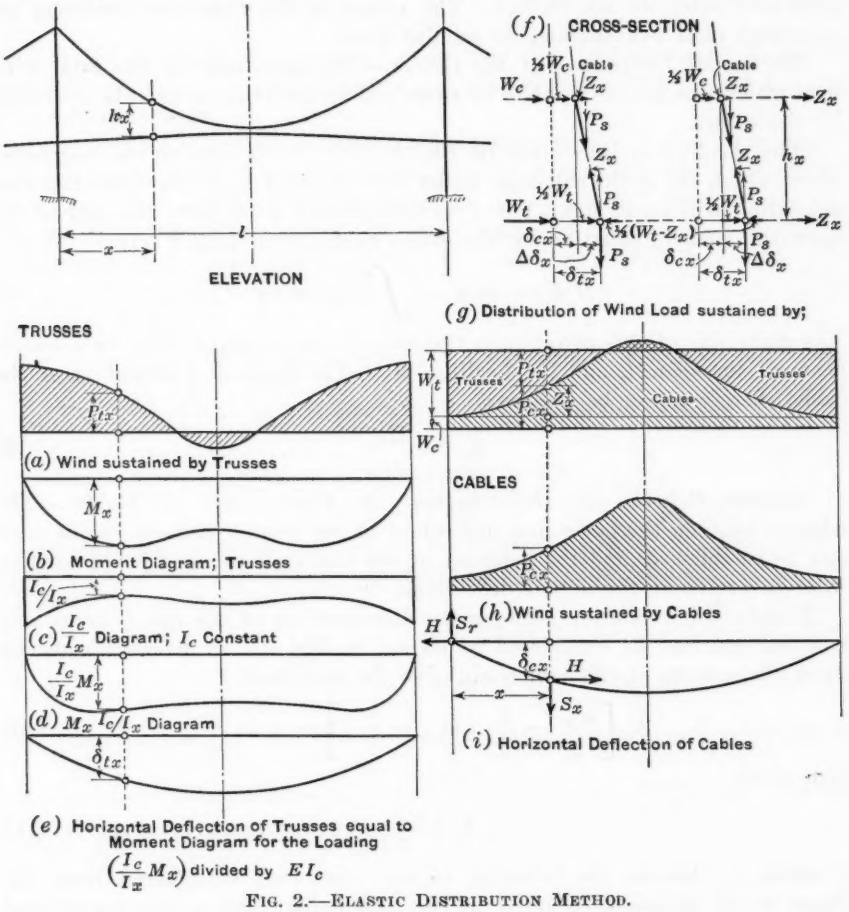

FIG. 2.—ELASTIC DISTRIBUTION METHOD.

The total wind load acting on the structure is plotted on a diagram consisting of w_c and w_t. At the towers, of course, the distribution of the wind load between cable and trusses is given. Along the span any distribution may be assumed. Such a probable distribution is indicated by the curve drawn on the load diagram. The wind load thus assumed, sustained by the trusses, is then plotted along the span on Fig. 2 (*a*). The moments for the load distribution shown in Fig. 2 (*a*) are then computed and plotted, as shown in Fig. 2 (*b*). The curve of the ratio of an assumed constant moment of inertia to the actual moment of inertia of the trusses is plotted along the span, as shown in Fig. 2 (*c*). The ordinates of Fig. 2 (*b*) and Fig. 2 (*c*) are

then multiplied by each other and plotted as in Fig. 2 (*d*). The products are divided by $E\ I_c$ and plotted as in Fig. 2 (*e*). The resulting ordinates in Fig. 2 (*e*) represent the horizontal deflections of the trusses in accordance with the arbitrary distribution assumed at the beginning.

Of course, the diagrams in Fig. 2 (*b*) to Fig. 2 (*e*) are merely illustrative and, as a rule, are not plotted. The values of the respective ordinates are computed most conveniently in tabular form.

Horizontal Deflections of the Cables.—The remainder of the total wind load which was not assumed to be sustained by the trusses, must be sustained by the cables.

This loading is illustrated in Fig. 2 (*h*). To determine the horizontal curve which the cable will take under this assumed load, the following consideration will be of use. For a section distant x ft. from the end of the span, the moment equation for the cables reads (from Fig. 2 (*i*)):

$$H\ \delta_{cx} = S_r\ x - \int_o^x p_{tx}\ x\ d_x \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

The right side of this equation is the same as the moment, M_{ox}, in a simple beam of the span, l, and the loading, p_{tx}. The horizontal deflection of the cable, therefore, can be written:

$$\delta_{cx} = \frac{M_{ox}}{H} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Relation Between the Deflections of the Trusses and the Cables.—The relation between the horizontal deflections of the trusses and the cables must now be established, and the behavior of the two systems must be equated in order to arrive at the actual deflections.

To establish a relation between the deformations of the two systems, any cross-section may be considered as shown in Fig. 2. A moment equation about the centers of the cables will give the equation:

$$h_x \left[\frac{w_{tx}}{2} - \frac{1}{2} \left(w_{tx} - z_x \right) \right] = \Delta\ \delta_x\ \frac{p_{sx}}{2} \quad \ldots\ldots\ldots\ldots (10)$$

from which,

$$\Delta\ \delta_x = \frac{h_x}{p_{sx}}\ z_x \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

in which, z_x denotes the intensity of the wind load transferred from the trusses to the cables. To arrive at the actual distribution, this transferred wind load, z_x, must be found by trial computations which will satisfy all three equations, (7), (9), and (11). The determination is made by the method of successive approximations.

The method can be applied for any loading condition, including live load, as well as wind pressure. It can also meet specifications which require the wind on the live load to be treated as a moving load in combination with the highest stresses caused by live load and temperature.

The solution may be made graphically or analytically. The analytical method, however, was found to be the more convenient if systematically treated; the results are also more accurate than those derived graphically.

Example of Wind Distribution in a 4 000-Foot Span.—To demonstrate the application of the Uniform Distribution Method and the Elastic Distribution Method, a numerical example of each will be given as applied to a suspension bridge with a span of 4 000 ft., for which a design has been made. It will be denoted as Proposed Design No. 2.

Example 1

Let l = 4 000 ft.; h_c (normal) = 500 ft.; dead load = 22 000 lb. per ft. of bridge; live load = 6 000 lb. per ft. of bridge; w_t = 1 150 lb. per ft.; and w_c = 150 lb. per ft.

The vertical deflection at mid-span for a 6 000-lb. live load, at the highest temperature (+ 40° Fahr. above normal) is 9.0 ft., downward. Therefore, h_c = 500 + 9 = 509 ft. and h_t = 509 + 10 = 519 ft. Furthermore, the horizontal cable pull equals H (for dead load and for live load at highest temperature) = 110 000 000 lb.; and I (constant) = 1 620 000 in.^{2} ft.^{2}

The wind load transferred from trusses to cables is determined by Equation (5); thus:

$$a = h_c\, l^2\, H = 509 \times 4\,000^2 \times 110\,000\,000 = 8.958 \times 10^{17}$$

$$b = 9.6\, h_t\, EI = 9.6 \times 29\,000\,000 \times 1\,620\,000 \times 519 = 2.340 \times 10^{17}$$

and,

$$z = \frac{8.958 \times 1\,150 - 2.340 \times 150}{8.958 \times 2.340} = 880 \text{ lb.}$$

Therefore, the wind load sustained by the trusses equals 1 150 − 880 = 270 lb. per ft.; and the wind load sustained by the cables equals 880 + 150 = 1 030 lb. per ft.

By Equation (1) the horizontal deflection of the cable at mid-span equals

$$\delta_c = \frac{1\,030 \times 4\,000^2}{8 \times 110\,000\,000} = 18.7 \text{ ft.; and } \delta_t = \frac{519}{509} \times \delta_c = 19.1 \text{ ft.}$$

The moment curve for the trusses is parabolic; for a loading of 270 lb., the mid-span moment is $M = \frac{270 \times 4\,000^2}{8} = 540\,000\,000$ ft-lb.; and the reactions of the wind forces, on one tower, are:

At the top,

$$R = 0.5\,(1\,030 \times 4\,000) = 2\,060\,000 \text{ lb.}$$

and at the elevation of the trusses,

$$R = 0.5\,(270 \times 4\,000) = 540\,000 \text{ lb.}$$

Example 2

In addition to the values in Example 1, let E = 29 000 kips per in.^{2}; $\lambda = \frac{4\,000}{20} = 200$ ft.; $\delta_{tx} = \frac{\lambda^4}{E I_c} M_{mx} = 0.03406\, M_{mx}$; $\delta_{cx} = \frac{\lambda^2}{H} M_{ox} = 0.3636\, M_{ox}$: and p_{sx} = live load (6 kips) plus dead load of suspended structure (16 kips), or 22 kips per ft. of bridge (constant).

TABLE 1.—ELASTIC DISTRIBUTION METHOD APPLIED TO PROPOSED DESIGN NO. 2

x, in feet	p_{lx}, in kips per foot	S'_x, in kips	M'_x, in kip-feet	$\frac{I_c}{I_x}$	$\frac{I_c}{I_x}M'_x$, in kip-feet	S_{mx}, in kip-feet^2	M_{mx}, in kip-feet^3	δ_{lx}, in feet	p_{cx}, in kips per foot	S_{ox}, in kips	M_{ox}, in kip-feet	δ_{cx}, in feet	h_x, in feet	$\frac{h_x}{p_{sx}}$, in kips per foot^2	Z_x, in kips per foot	$\Delta\delta_x = \frac{h_x}{p_{sx}} Z_x$, in feet	$\Delta\delta_x = \delta_{lx} - \delta_{cx}$ in feet	S_x, in kips	M_x, in 1 000 kip-ft.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
		4.790				106.58				7.560								958	
0.05 l	1.105		4.790	1.0	4.790		106.58	3.6	0.195		7.560	2.7	435.2	19.78	0.045	0.9	0.9		192
		3.685				101.78				7.365								737	
0.10 l	1.045		8.475	1.0	8.475		208.36	7.1	0.255		14.925	5.4	346.0	15.73	0.105	1.7	1.7		339
		2.640				93.31				7.110								528	
0.15 l	0.970		11.115	1.0	11.115		301.67	10.3	0.330		22.035	8.0	267.2	12.15	0.180	2.2	2.3		445
		1.670				82.19				6.780								334	
0.20 l	0.860		12.785	1.0	12.785		383.86	13.1	0.440		28.815	10.5	199.0	9.05	0.290	2.6	2.6		511
		0.810				69.41				6.340								162	
0.25 l	0.740		13.595	1.0	13.595		453.27	15.4	0.560		35.155	12.8	141.2	6.42	0.410	2.6	2.6		544
		0.070				55.81				5.780								14	
0.30 l	0.600		13.665	1.0	13.665		509.08	17.3	0.700		40.935	14.9	94.0	4.27	0.550	2.3	2.4		547
		—0.530				42.15				5.080								—106	
0.35 l	0.360		13.135	1.0	13.135		551.23	18.8	0.940		46.015	16.7	57.2	2.60	0.790	2.0	2.1		525
		—0.890				29.01				4.140								—178	
0.40 l	0.035		12.245	1.0	12.245		580.24	19.8	1.265		50.155	18.2	31.0	1.41	1.115	1.6	1.6		490
		—0.925				16.77				2.875								—185	
0.45 l	—0.500		11.320	1.0	11.320		597.01	20.3	1.800		53.030	19.3	15.2	0.69	1.650	1.1	1.0		453
		—0.425				5.45				1.075								—85	
0.50 l	—0.850		10.895	1.0	10.895		602.46	20.5	2.150		54.105	19.7	10.0	0.45	2.000	0.9	0.8		436

TABLE 2.—COMPARISON OF DESIGN CHARACTERISTICS

Item	Description	Unit	Proposed Design No. 2 (Fig. 3)	Manhattan Bridge (Fig. 4)	Delaware River Bridge (Fig. 5)	Detroit River Bridge (Fig. 6)	Proposed Design No. 1 (Fig. 7)	George Washington Bridge (Fig. 8)
1	Span length	Feet	4 000	1 470	1 750	1 840	2 640	3 500
2	Dead load of cables	Pounds*	6 000	4 280	4 200	1 600	4 000	13 000
3	Dead load of suspended structure	Pounds*	16 000	19 000	21 800	11 800	12 000	26 000
4	Total dead load	Pounds*	22 000	23 280	26 000	12 400	16 000	39 000
5	Live load	Pounds*	6 000	16 000	12 000	0	6 000	4 000
6	Dead load plus live load	Pounds*	28 000	39 280	38 000	12 400	22 000	43 000
7	Horizontal cable pull, H, due to dead and live load and temperature change	Thousand pounds	110 000	63 800	69 060	25 800	71 240	194 950
8	Horizontal Wind Pressure: w_t, on trusses	Pounds*	1 150	1 800	1 200	670	1 150	1 200
9	w_c, on cables	Pounds*	150	200	150	160	150	200
10	Transferred wind load from trusses to cables, z	Pounds*	880	264	306	255	618	1 060
	Lateral Deflections in Trusses at Mid-Span by Elastic Distribution:							
11	Cable not resisting	Feet	82.0	2.4	3.4	11.4	20.8	90
12	Cable resisting	Feet	20.5	2.06	2.6	7.2	10.1	11.4
13	Percentage reduction in lateral deflection because of cable action		75	12	23	37	51	87
14	Deflection in cables	Feet	19.7	1.97	2.52	6.9	9.7	10.3
	Deflection by Uniform Distribution:							
15	Trusses	Feet	19.1	2.03	2.58	7.0	9.6	10.3
16	Cables	Feet	18.7	1.96	2.52	6.8	9.4	9.9

* Pounds per foot of bridge.

The application of the Elastic Distribution Method to this design and the analytical procedure followed, is shown in Table 1; it is illustrated diagrammatically in Fig. 3. A comparison of design characteristics is given in Table 2.

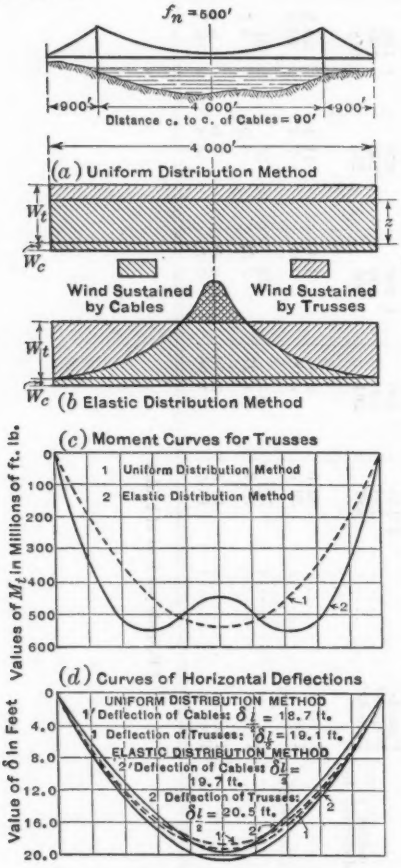
f_n = 500′
900′
4 000′
900′
Distance c. to c. of Cables = 90′
4 000′
(a) Uniform Distribution Method
W_t
W_c
z
Wind Sustained by Cables
Wind Sustained by Trusses
(b Elastic Distribution Method
(c) Moment Curves for Trusses
1 Uniform Distribution Method
2 Elastic Distribution Method
Values of M_x in Millions of ft. lb.
(d) Curves of Horizontal Deflections
UNIFORM DISTRIBUTION METHOD
1′ Deflection of Cables: $\delta\frac{l}{2}$ = 18.7 ft.
1 Deflection of Trusses: $\delta\frac{l}{2}$ = 19.1 ft.
ELASTIC DISTRIBUTION METHOD
2′ Deflection of Cables: $\delta\frac{l}{2}$ = 19.7 ft.
2 Deflection of Trusses: $\delta\frac{l}{2}$ = 20.5 ft.
Value of δ in Feet

FIG. 3.—PROPOSED DESIGN NO. 2.

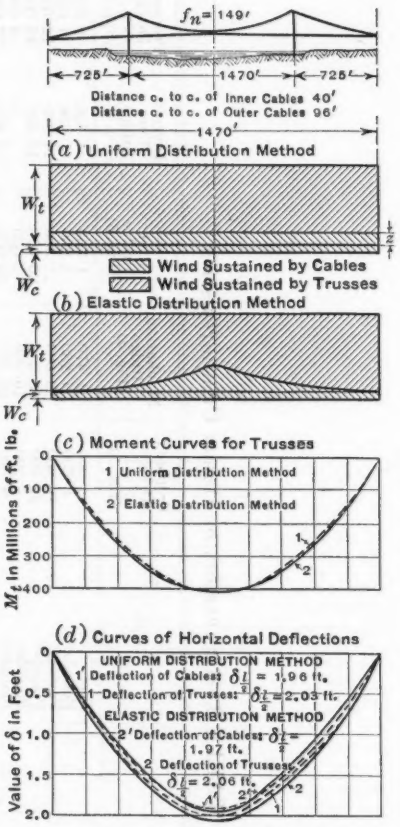
f_n = 149′
725′
1470′
725′
Distance c. to c. of Inner Cables 40′
Distance c. to c. of Outer Cables 96′
1470′
(a) Uniform Distribution Method
W_t
W_c
z
Wind Sustained by Cables
Wind Sustained by Trusses
(b) Elastic Distribution Method
(c) Moment Curves for Trusses
1 Uniform Distribution Method
2 Elastic Distribution Method
M_x in Millions of ft. lb.
(d) Curves of Horizontal Deflections
UNIFORM DISTRIBUTION METHOD
1′ Deflection of Cables: $\delta\frac{l}{2}$ = 1.96 ft.
1 Deflection of Trusses: $\delta\frac{l}{2}$ = 2.03 ft.
ELASTIC DISTRIBUTION METHOD
2′ Deflection of Cables: $\delta\frac{l}{2}$ = 1.97 ft.
2 Deflection of Trusses: $\delta\frac{l}{2}$ = 2.06 ft.
Value of δ in Feet

FIG. 4.—MANHATTAN BRIDGE.

The wind reactions for one tower are found to be: At the top,

$$R = 7.56 \times 200 + 0.15 \times 100 = 1\,527 \text{ kips}$$

and, at the elevation of the trusses,

$$R = 958 + 1.15 \times 100 = 1\,073 \text{ kips}$$

At first, the wind load, z_x, transferred from the trusses to the cables was assumed, and the computation was made on the basis of this guess of distribution. The assumption of z_x must be changed and the computation done over again until the differences in the horizontal deflections between trusses and cables, as determined by Equations (7) and (8), are practically equal to the differences computed by Equation (11). The span of 4 000 ft. was divided into twenty equal parts, each part having a length, $\lambda = 200$ ft. The trans-

ferred wind load, z_x, was assumed and thereby the loads sustained by the trusses and by the cables were fixed. Table 1 contains twenty columns which are explained as follows:

Column (1) = distance, x, in feet, from end of span.
Column (2) = p_{tx}, wind pressure sustained by trusses, in kips per foot of bridge.
Column (3) = S'_x, shear for loading, p_t, in kips, reduced by λ.
Column (4) = M'_x, moment for loading, p_t, in kip-feet, reduced by λ^2.
Column (5) = $\frac{I_c}{I_x}$, ratio of moments of inertia for the various sections of the trusses; I_c is constant.
Column (6) = $\frac{I_c}{I_x} M_x$, in kip-feet, reduced by λ^2.
Column (7) = S_{mx}, shear, in kip-feet, for loading, $\frac{I_c}{I_x} M'_x$, reduced by λ^3.
Column (8) = M_{mx}, moment, in kip-feet^3, for loading, $\frac{I_c}{I_x} M'_x$, reduced by λ^4.
Column (9) = δ_{tx}, horizontal deflection of trusses, in feet, = $\frac{\lambda^4}{E I_c} M_{mx}$ = values in Column (8) multiplied by $\frac{\lambda^4}{E I_c}$.
Column (10) = p_{cx}, wind pressure sustained by cables, in kips per foot of bridge.
Column (11) = S_{ox}, shear for loading, p_{cx}, in kips, reduced by λ.
Column (12) = M_{ox}, moment for loading, p_{cx}, in kip-feet, reduced by λ^2.
Column (13) = δ_{cx}, horizonttal deflection of cables, in feet = $\frac{\lambda^2}{H} M_{ox}$ = the values in Column (12) multiplied by $\frac{\lambda^2}{H}$.
Column (14) = h_x, hanger length, in feet.
Column (15) = $\frac{h_x}{p_{sx}}$, in kip-feet^2; p_{sx} is sum of dead load of suspended structure and live load on it, in kips per foot of bridge.
Column (16) = z_x, transferred wind load from trusses to cables, in kips per foot of bridge.
Column (17) = $\Delta\delta_x = \frac{h_x}{p_{sx}} z_x$, difference in horizontal deflections between trusses and cables.
Column (18) = $\Delta\delta_x = (\delta_{tx} - \delta_{cx})$, difference in horizontal deflections between trusses and cables.
Column (19) = S_x, shear in trusses, in kips, equal to S_x in Column (2) multiplied by λ.
Column (20) = M_x, moment in trusses, in kip-feet, equal to M_x in Column (3) multiplied by λ^2.

The assumptions of z_x should be changed until the values of $\Delta\delta_x$ in Column (17) and Column (18) coincide.

Comparative Results of Six Bridges

Six bridges of various span lengths and various loads have been investigated by both the Uniform Distribution Method and the Elastic Distribution Method. The two methods do not give decided differences in moments for the shorter span bridges. For the Manhattan Bridge (Fig. 4) and the Delaware River Bridge (Fig. 5), the Uniform Distribution Method gives slightly

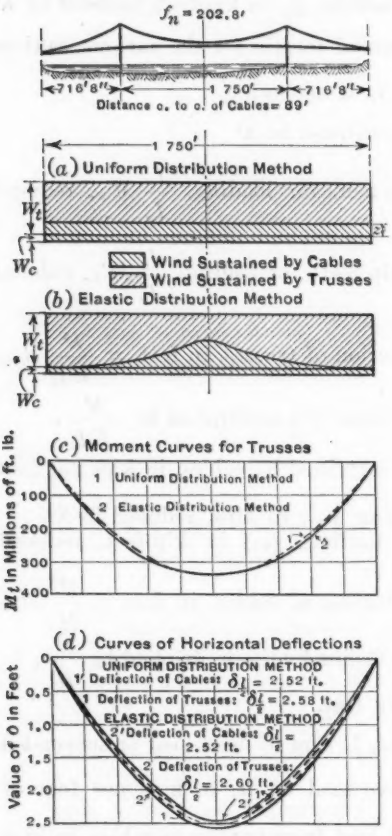
f_n = 202.8′
716′8″
1 750′
716′8″
Distance c. to c. of Cables = 89′
1 750′
(a) Uniform Distribution Method
W_t
W_c
Wind Sustained by Cables
Wind Sustained by Trusses
(b) Elastic Distribution Method
(c) Moment Curves for Trusses
M_t in Millions of ft. lb.
1 Uniform Distribution Method
2 Elastic Distribution Method
(d) Curves of Horizontal Deflections
Value of δ in Feet
UNIFORM DISTRIBUTION METHOD
1′ Deflection of Cables: $\delta\frac{l}{2}$ = 2.52 ft.
1 Deflection of Trusses: $\delta\frac{l}{2}$ = 2.58 ft.
ELASTIC DISTRIBUTION METHOD
2′ Deflection of Cables: $\delta\frac{l}{2}$ = 2.52 ft.
2 Deflection of Trusses: $\delta\frac{l}{2}$ = 2.60 ft.

Fig. 5.—Delaware River Bridge.

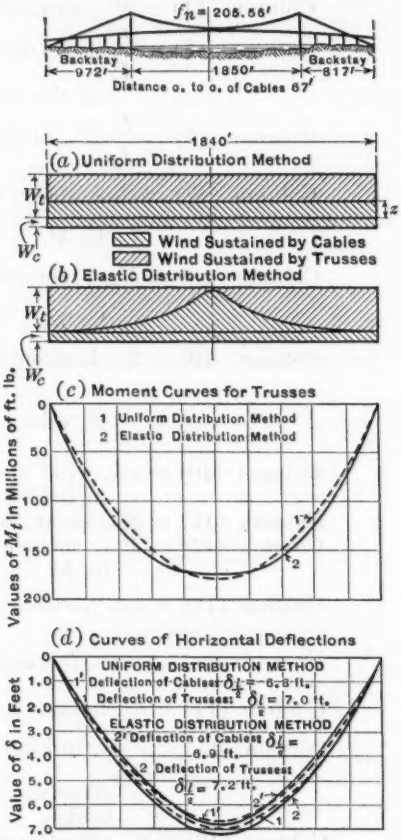
f_n = 205.56′
Backstay
972′
1850′
Backstay
817′
Distance c. to c. of Cables 67′
1840′
(a) Uniform Distribution Method
W_t
W_c
Wind Sustained by Cables
Wind Sustained by Trusses
(b) Elastic Distribution Method
(c) Moment Curves for Trusses
Values of M_t in Millions of ft. lb.
1 Uniform Distribution Method
2 Elastic Distribution Method
(d) Curves of Horizontal Deflections
Value of δ in Feet
UNIFORM DISTRIBUTION METHOD
1′ Deflection of Cables: $\delta\frac{l}{2}$ = 6.8 ft.
1 Deflection of Trusses: $\delta\frac{l}{2}$ = 7.0 ft.
ELASTIC DISTRIBUTION METHOD
2′ Deflection of Cables: $\delta\frac{l}{2}$ = 6.9 ft.
2 Deflection of Trusses: $\delta\frac{l}{2}$ = 7.2 ft.

Fig. 6.—Detroit River Bridge.

lower values, with the exception at mid-span, where both curves coincide. The difference, however, does not exceed 2.2% at quarter-span for the Manhattan Bridge, and 5.2% for the Delaware River Bridge, taking the Elastic Distribution Method at 100 per cent.

With increasing spans the difference in the results of the two methods begins to grow. Toward the ends of the trusses the results of the Uniform Distribution Method are lower than those derived from the Elastic Distribution Method, whereas the opposite is true for the center portion. For the Detroit River Bridge (Fig. 6), the Uniform Distribution Method gives 8% less at quarter-span and 1.7% more at mid-span than the Elastic Distri-

bution Method. The corresponding values for "Proposed Design No. 1" (Fig. 7), George Washington Bridge (Fig. 8), and "Proposed Design No. 2" (Fig. 3), are 13%, 37%, and 26% less at quarter-span, respectively, and 5%, 61%, and 24% more at mid-span.

It is of interest to know to what extent the cables help to relieve the stresses in the trusses. Introducing the moment value in an unrestrained

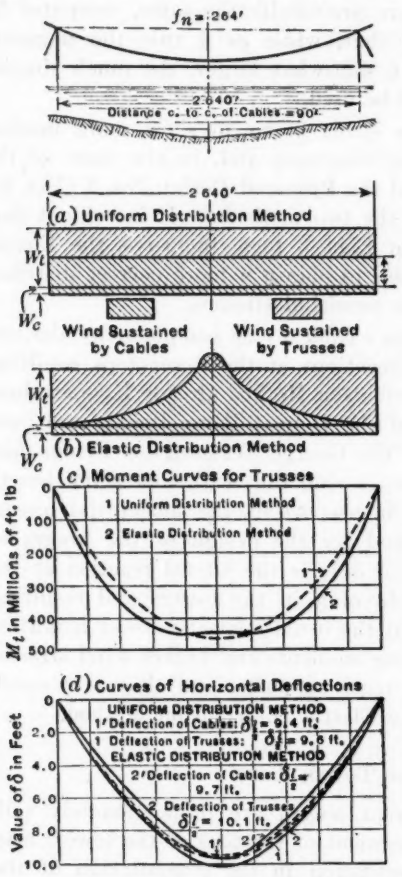

FIG. 7.—PROPOSED DESIGN NO. 1.

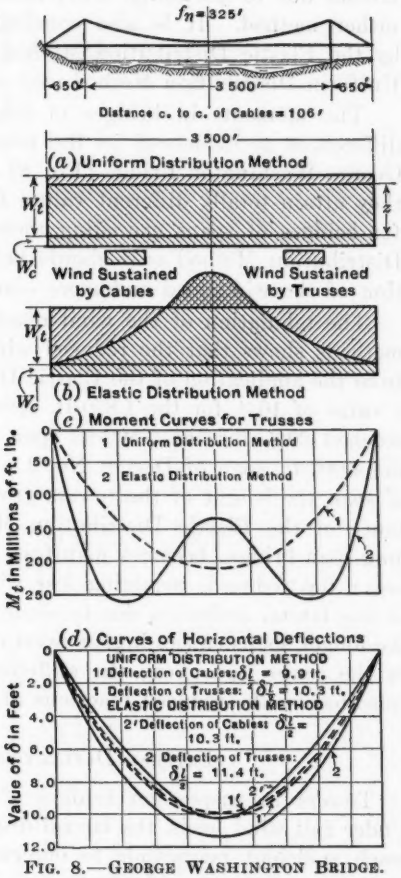

FIG. 8.—GEORGE WASHINGTON BRIDGE.

truss as 100%, not suspended from the cables, the Elastic Distribution Method gives for the various bridges:

Bridge	Quarter-span (Percentage)	Mid-span (Percentage)
Manhattan	87	85
Delaware	79	74
Detroit	67	61
Design No. 1	53	44
George Washington	19	7.2
Design No. 2	32	19

Attention is drawn to the fact that in all cases the Elastic Distribution Method gives lower wind reactions at the tops of the towers. Table 3 gives a comparison of the wind reactions on the towers by the two methods.

From the study of the moment curves of the various bridges, it may be concluded that for spans up to 1 500 ft. of moderate live-load capacity and up to 1 750 ft. of large live-load capacity, the horizontal moments in the trusses due to horizontal wind loads are practically the same, computed by either method. It is also conclusive that, while as a rule the moments by the Elastic Distribution Method are somewhat larger, the much simpler Uniform Distribution Method may well be used.

The moments in bridges of longer spans begin to show more marked differences as evidenced in the moment diagrams and, in the cases of the George Washington Bridge (Fig. 8) and the Proposed Design No. 2 (Fig. 3), they attain totally different values for the two methods. It is evident that for bridges of spans exceeding those in Figs. 3 to 8, inclusive, the Elastic Distribution Method only should be used to obtain a more correct distribution of the wind load and more correct resulting stresses.

The tabulation of the wind reactions (Table 3) as computed by the two methods shows that the reduced wind reactions at the tower tops resulting from the application of the Elastic Distribution Method already have attained a value of 10% for the 1 850-ft. span of the Detroit River Bridge and have reached 20% for the 3 500-ft. span of the George Washington Bridge and the 4 000-ft. span of Design No. 2. Considering that with increasing length of span the height of the towers grows in proportion, the substantial importance of the Elastic Distribution Method for the design of the towers of long-span bridges becomes manifest. Not only is the lateral reaction at the tower top reduced, requiring less wind-bracing in the towers and resulting in less lateral deflection due to shear, but the usually greater tower height of the longer spans causes higher overturning moments and higher wind stresses in the tower legs above the stiffening truss supports, and these are much benefited by the reduced reactions of the Elastic Distribution Method.

Lateral Deflection of Tower Tops

Towers in suspension bridges of great length are proportionately tall. Under full wind loads, the lateral displacement of the tops of the towers may reach sufficient magnitude to deserve attention in the computation of the wind stresses in the stiffening truss. In the case of the Golden Gate Bridge, with a span of 4 200 ft., these displacements are about 16 in. The lateral displacements of the stiffening truss, as well as its bending moment, will be increased thereby. In such cases a computation should be made to determine the additional stresses involved.

Comparative Lateral Stiffness of Bridges

The stability and lateral stiffness of a bridge depend on its width. General bridge specifications usually prescribe a minimum for the ratio of span to width. Even for spans of moderate lengths the lateral stiffness of the

TABLE 3.—COMPARISON OF WIND REACTIONS

Item	Description	Uniform Distribution		Elastic Distribution		Percentage difference	Uniform Distribution		Elastic Distribution		Percentage difference
		Kips	Percentage	Kips	Percentage		Kips	Percentage	Kips	Percentage	
		Manhattan Bridge					Delaware River Bridge				
1	Top	341	23	281	19	4	399.0	34	305.35	26	8
2	Truss level	1 129	77	1 189	81	4	782.25	66	875.9	74	8
3	Total	1 470	100	1 470	100	.....	1 181.25	100	1 181.25	100	.....
4	Transferred	194	13.5	144	9.5	4	268	23	174.35	15	8
		Detroit River Bridge					Proposed Design No. 1				
5	Top	382	50	304	40	10	1 014	59	746	43	16
6	Truss level	382	50	460	60	10	702	41	970	57	16
7	Total	764	100	764	100	.....	1 716	100	1 716	100	.....
8	Transferred	235	31	157	21	10	816	48	548	32	16
		George Washington Bridge					Proposed Design No. 2				
9	Top	2 205	90	1 697	69	21	2 060	79	1 527	59	20
10	Truss level	245	10	753	31	21	540	21	1 073	41	20
11	Total	2 450	100	2 450	100	.....	2 600	100	2 600	100	.....
12	Transferred	1 855	76	1 347	55	21	1 760	68	1 227	48	20

trusses already begins to claim consideration in the determination of the width. For long-span bridges the required width between trusses raises a serious problem.

In suspension bridges the center of gravity is so low below the points of support that the suspended structure is always in stable equilibrium and the lateral stability of the towers only has to be considered. The study of the behavior of suspension bridges under the action of lateral loads has also disclosed that they are relatively stiffer than simple trusses of the same moments of inertia and that their relative stiffness increases with the length of the span.

Table 2, Items 11, 12, and 13, showing the lateral deflections at mid-span of the bridges studied in this paper, will serve to demonstrate the relative stiffness of simple to suspended trusses.

The effect of the cable in reducing the lateral deflection of the trusses is so great that in the longer spans the resulting deflection is one-half, one-quarter and one-eighth of those which would result without the action of the cables. It can be stated, in other words, that long-span suspension bridges are two to four times laterally stiffer than simple spans and arches of the same width and chord area. This characteristic of long-span suspension bridges is of the utmost importance in discussing the required width between trusses.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

CONSTRUCTION METHODS ON THE WELLAND SHIP CANAL[1]

BY E. G. CAMERON,[2] ESQ.

SYNOPSIS

The growth of the Welland Canal of 1829 to the Welland Ship Canal of 1931 represents a vast change in construction methods. Of the total cost of $130 000 000, about 60% was for earth and rock excavation and concrete. A description of the methods used in handling these three items of construction constitutes the body of the paper.

The letting of contracts, the division of the work into sections, the labor force and rates, and the difficulties and delays occasioned by the World War, are discussed preliminary to methods and costs.

In the study of costs it is to be borne in mind that they represent two periods of markedly different dollar values, namely, pre-war and post-war. The writer has provided extensive data showing the wide range of contract prices, but has confined his information to field costs, leaving the reader to use his own preference in adding overhead and rental charges.

Labor problems, wages paid, machine and plant outputs, together with descriptions of hydraulic dredging, embankment, and breakwater construction, rock drilling and removal methods, will be of interest to contracting engineers generally.

That section of the paper devoted to concrete and a description of the plant layouts, forms, and the methods of constructing reinforced concrete cribs afloat, and their various uses in structures, should be of particular interest to those who specialize in concrete.

The final section of the paper, devoted to the construction of a monolithic concrete siphon culvert, in which six tubes, each 20 ft. in diameter, carry

NOTE.—Written discussion on this paper will be closed in **August, 1932**, *Proceedings*.

[1] Presented at the Joint Meeting of the Construction and Waterways Divisions, Milwaukee, Wis., July 11, 1929.

[2] Prin. Asst. Engr., Welland Ship Canal, St. Catharines, Ont., Canada.

the waters of Chippawa Creek under the 30-ft. draft prism of the canal, treats of the construction and use of a large cellular steel sheet-pile cofferdam and the methods and difficulties experienced in carrying excavation and concrete operations in steel sheet-pile trenches sunk to a depth of 80 ft. in very soft clay.

History

Nature imposed an almost unsurmountable obstacle to the development of an inland waterway from the head of the Great Lakes to the Atlantic seaboard, by placing Lake Erie at an elevation of 525½ ft. above the next link in the chain—Lake Ontario.

From the days of Champlain (1600) until early in the Nineteenth Century, the limited but ever-growing commerce of the Great Lakes was portaged across the Niagara Peninsula from Queenston, the limit of navigation in the Lower Niagara River, to Chippawa, Ont., Canada, where the Upper Niagara was again navigable through to Lake Erie.

On November 30, 1824, a private company largely financed by English and American capital, began the construction of a canal to provide a waterway with an 8-ft. draft between the two lakes. The route followed the winding valley of the Twelve-Mile Creek from its outlet into Lake Ontario at Port Dalhousie to its source in the face of the Niagara Escarpment in the vicinity of Thorold, Ont. (See Fig. 1.) From this point, at the upper plateau level of the Peninsula, an entirely artificial channel was excavated due south to Port Robinson, Ont.—midway between the lakes. Chippawa Creek and the Upper Niagara River were used from Port Robinson to Lake Erie.

First Canal.—The construction of this first Welland Canal, using 40 wooden locks, each 110 ft. long by 22 ft. wide, was entirely effected by pick and shovel, supplemented at the lock structures by the broad-axe. The only means of transporting the excavation was by wheel-barrow. The rather meager record of this early undertaking registers the fact that, as a result of advertising a reward of £5 for the best mechanical contrivance for the removal of the excavation from the prism, a windlass was perfected by means of which the loaded wheel-barrows were hauled out of the prism and to the top of the adjacent spoil banks. Immigrant labor, very largely originating from Ireland, provided the working force on construction, and the rate of pay was $0.63 per day.

On November 30, 1829, just five years after the first sod was turned, two vessels—the *Ann and John,* of Toronto, Ont., Canada, and the *R. H. Broughton,* of Youngstown, N. Y.—passed through the completed canal from Lake Ontario to Lake Erie. In the first decisive engagement between Nature and Man, the latter had come off victorious.

Although the Erie Canal, across New York State to the Hudson River, had been previously completed, the combined accommodation of the two canals rapidly became inadequate to meet the increasing demands of shipping.

Second Canal.—In 1841 the Government of Upper Canada purchased this first canal from the Company, and immediately replaced it by a new Welland Canal of 9-ft. draft. This canal followed the same route; it had 27 locks of masonry construction, each 150 ft. long by 26.5 ft. wide. A few years later this second canal was deepened to 10-ft. draft and extended through to Lake Erie. The lock section of this canal is still intact and the splendid stonework

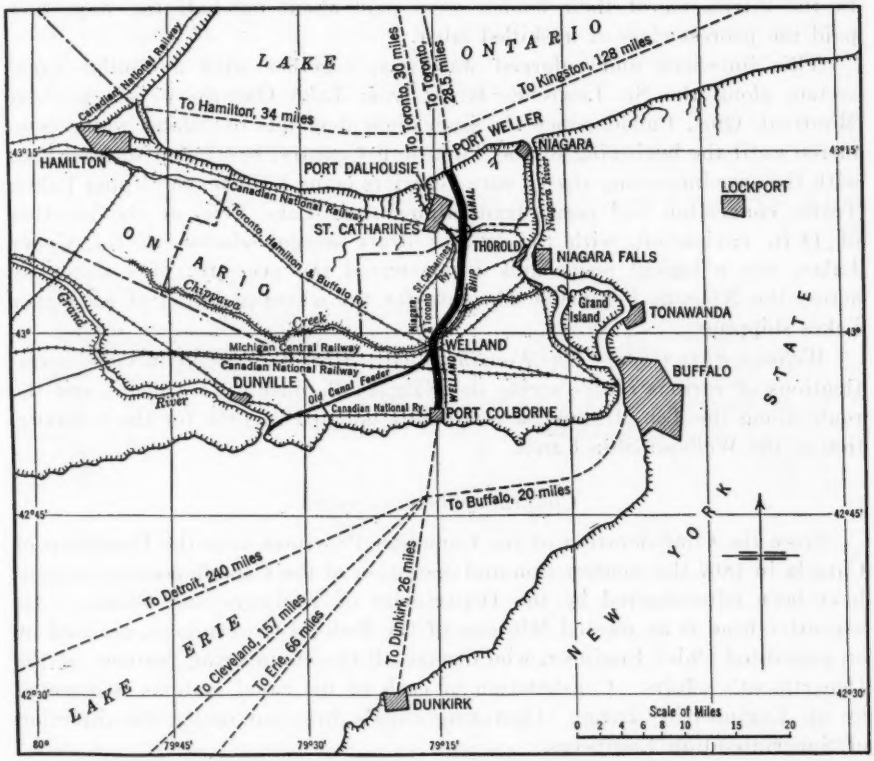

FIG. 1.—MAP OF NIAGARA PENINSULA, SHOWING ROUTE OF WELLAND SHIP CANAL.

and its remarkable state of preservation bear witness to the skill and integrity of the artisans of nearly a century ago.

Third Canal.—The construction of the third Welland Canal (which was entirely replaced by the Welland Ship Canal in 1930), was inaugurated in 1870 by the Dominion of Canada. By 1887 a waterway, providing a 14-ft. draft, with 25 locks, each 270 ft. long by 45 ft. wide, was available for shipping. This was a much straighter and more direct route than that followed by the preceding canals across the lower plateau of the Peninsula to the foot of the Escarpment. On reaching the upper plateau level, much the same alignment as the previous canals was followed to Port Colborne, Ont.

As compared with present-day construction practices, comparatively primitive methods were utilized even on the third Welland Canal. The pick and shovel still held sway, assisted by teams and scrapers, while at the lock struc-

tures, derricks (chiefly hand-operated) were the only mechanical means then available for handling the masonry. A small steam shovel was introduced on the work toward the end of this construction period. However, as this type of equipment was then only in the development stage, the machine proved more of a novelty and an object of curiosity than an assistance in the actual execution of the work. Skilled artisans, thoroughly schooled in the intricacies of their trades, were paid about one-half the wage now paid the poorest class of unskilled labor.

This improved and enlarged waterway, together with a similar canal system along the St. Lawrence River from Lake Ontario to sea level at Montreal, Que., Canada, met the immediate demands of inland water commerce until the beginning of the Twentieth Century, but failed to keep pace with the ever-increasing size of cargo carriers being built on the Upper Lakes. Traffic congestion and cargo transshipment on Lake Erie, at the junction of 14-ft. navigation, with the greater draft accommodation of the Upper Lakes, was a logical result and demonstrated the necessity of a waterway across the Niagara Peninsula adequate for the accommodation of all Upper Lakes shipping.

Welland Ship Canal—The Fourth Canal.—With this object in view, investigations of various routes across the Niagara Peninsula were made, and the route along the Ten-Mile Creek was finally adopted in 1913 for the construction of the Welland Ship Canal.

Construction Contracts

Since the Confederation of the Canadian Provinces into the Dominion of Canada in 1867 the construction and operation of the Canadian canal systems have been administered by the Department of Railways and Canals. Its executive head is an elected Minister of the Federal Government, assisted by an appointed Chief Engineer, who directs all the engineering features of the Department's affairs. Construction of each of the canal systems is directed by an Engineer-in-Charge. Operating canals function under the direction of Superintending Engineers.

Method of Letting Contracts.—In initiating the construction of the Welland Ship Canal, the Department of Railways and Canals adhered to its standard practice of having the work done by unit price contracts.

The magnitude of the work and the construction difficulties involved made it necessary to adopt a procedure that would insure the work being placed in the hands of only thoroughly experienced and financially responsible contractors. To effect this, the entire work was divided into sections, each with an estimated cost considerably in excess of $1 000 000. In order to qualify, bidders were required to deposit security to the amount of 5% of their tenders. The contract for each division of the work was awarded to the bidder who, possessing the necessary credentials of experience and financial ability, submitted the lowest tender.

In the actual execution of the work the contractor was permitted to enlist the assistance of smaller competent contractors, by sub-letting portions of

his division to parties who, in turn, could satisfactorily establish their ability to do the work allotted to them. In this case, however, the general contractor was free to make his own selection quite irrespective of whether it was based on the lowest sub-tender prices for the work.

That the foregoing procedure is an entirely satisfactory and effective method of handling the construction of such a large undertaking is best indicated by the fact that, while unit prices prevailing were low when the work was done, in no case during the entire period of construction did a general contractor or any of his sub-contractors fail to carry through his work in a manner satisfactory to the Department.

At the inception of construction in 1915 the work was divided, for the purpose of awarding contracts, into nine sections as shown in Fig. 2. Section

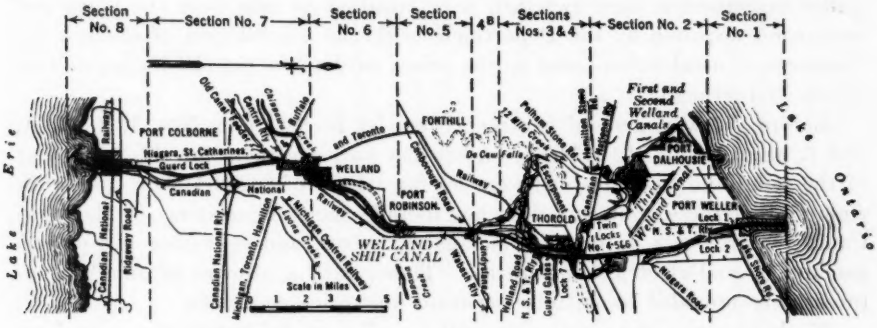

FIG. 2.—PRISM SECTIONS, WELLAND SHIP CANAL

No. 1 embraced the work at the north or Lake Ontario end and Section No. 9 incorporated the Port Colborne Harbor work at the south, or Lake Erie, end of the canal. Later, the number of sections was reduced to eight by combining the land and harbor portions of the south end of the canal at Port Colborne.

Sections Nos. 1, 2, and 3 comprised the much heavier lock division of the canal from Lake Ontario to the summit, or Lake Erie, level. With a view to making the completion of all parts of the canal synchronize these sections were first let by contract and work was actually begun in the fall of 1913, together with Section No. 5—the northerly part of the summit level.

Delay Due to World War.—The outbreak of the World War in 1914 soon had its effect on construction activities on these sections of the work. By the spring of 1916 both labor and materials had become so scarce from diversion for war purposes, and the financial burden of the war had become so heavy on Canada, that the complete suspension of construction became necessary and the original contracts were cancelled.

The Department paid each contractor in full for all work done, and took over all plant at its cost on the work, less a percentage for depreciation equivalent to the percentage of the entire contract value actually done at the time of cancellation. Expenditures on the Welland Ship Canal up to the time all work was suspended approximated $16 000 000.

Post-War Activities.—Construction operations were completely suspended until the spring of 1919, when, as a means of furnishing employment to the many men returning again to civil life after the unsettling effects of the war, work was resumed on Sections Nos. 1, 2, 3, and 5, on a "cost-plus" basis, by the original contractors.

While this arrangement met the immediate object for which it was devised, it created other conditions that greatly retarded the progress of the work and so augmented the cost that the Department discontinued the percentage contracts in 1921 and thereafter adhered to the unit-price basis under which the remainder of the work was contracted. The partly completed Sections Nos. 1 and 2 were combined under one contract, as were also Sections Nos. 3 and 4. The contractors to whom the sections previously under construction were awarded, were required to take over the plant and equipment acquired by the Department with the cancellation of the pre-war contracts, at a valuation based on the prices paid at the time of its acquisition by the Department.

Before the war, general contracts were let for Sections Nos. 1, 2, 3, 4A, and 5, aggregating $12 017 000. After the war, contracts for the remainder of the canal and the gate-yard were let for a total of $62 932 000. The work embraced in these contracts does not include such major items of the Ship Canal requirements as bridges, lock-gates, valves, and operating equipment, machinery, and electrical apparatus. These, with a number of smaller features, were provided by entirely separate unit-price contracts.

Working plant and equipment with a value of $8 730 000 was placed on the various divisions of the work, including the equipment acquired on Sections Nos. 1, 2, 3, 4, and 5, as previously explained.

Labor Rates

With modern and up-to-date equipment available, labor was still the governing factor of the construction operations on the Welland Ship Canal. In all public works constructed by the Dominion of Canada, the Federal Department of Labor exercises jurisdiction in the matter of fixing the minimum rates which must be paid the various classes employed.

At the inception of construction on any public work the Department of Labor investigates conditions prevailing in the immediate area of such work, and from the data thus obtained compiles a schedule of minimum wage rates which it requires shall be paid labor on the public work in question. This schedule of rates is furnished the parties tendering for their guidance in estimating costs, but is subject to revision by the Labor Department should conditions change during the period of construction.

The Department of Labor applied the foregoing procedure to this work, and the minimum rates of the principal classes of labor, their year by year fluctuations, and the application of overtime thereto, are given in Table 1. Only in a few exceptional cases did the contractors pay in excess of these minimum rates.

In cases of dispute between the contractor and his employees as to the fairness of rates being paid, such controversies were referred to the Federal Department of Labor which, after investigating the circumstances, would establish rulings governing the conditions in dispute, and both the contractor and his employees were obligated to abide by such prescribed regulations.

TABLE 1.—MINIMUM FAIR WAGE SCHEDULE IN EFFECT ON THE WELLAND SHIP CANAL

Class of labor	1913–16	1919–20	1921	1922	1923–24	1925–26	1927–30
Laborers	$0.20	$0.40	$0.375	$0.35	$0.40	$0.35	.....
Expert laborers	0.20	0.45	0.45	0.45	0.50	0.50	.....
Handymen	0.25	0.50	0.48	0.45	0.50	0.45	.....
Concrete finishers	0.25	0.50	0.52	0.50	0.55	0.50	.....
Carpenters	0.35	0.70	0.70	0.70	0.80	0.80	$0.85
Carpenter helpers	0.20	0.50	0.50	0.45	0.50	0.45	.....
Locomotive runners	0.38	0.60	0.60	0.60	0.65	0.60	.....
Firemen, locomotive crane	0.21	0.52	0.45	0.45	0.50	0.50	.....
Brakemen	0.24	0.50	0.50	0.45	0.50	0.45	.....
Conductors	0.36	0.60	0.60	0.60	0.65	0.60	.....
Shovel runners	0.48	0.87	0.87	0.87	0.92	0.92	1.00
Shovel cranesmen	0.345	0.68	0.68	0.68	0.73	0.73	0.80
Shovel firemen	0.20	0.52	0.52	0.50	0.55	0.55	0.65
Dragline operators	0.35	0.56	0.56	0.50	0.55	0.45	.....
Car repairers	0.20	0.55	0.56	0.50	0.55	0.45	.....
Blacksmiths	0.30	0.70	0.70	0.60	0.65	0.60	.....
Blacksmith helpers	0.20	0.50	0.50	0.45	0.50	0.45	.....
Boiler-makers	0.35	0.75	0.75	0.60	0.65	0.60	.....
Boiler-maker helpers	0.20	0.50	0.50	0.45	0.50	0.45	.....
Machinists	0.30	0.70	0.72	0.65	0.70	0.65	0.70
Machinist helpers	0.20	0.50	0.50	0.45	0.50	0.45	.....
Pipe fitters	0.25	0.60	0.70	0.70	0.75	0.65	.....
Riggers	0.30	0.65	0.65	0.65	0.65	0.65	.....
Rigger helpers	0.20	0.50	0.50	0.50	0.50	0.45	.....
Drill runners	0.225	0.55	0.52	0.45	0.50	0.50	.....
Powdermen	0.30	0.40	0.60	0.45	0.50	0.45	.....
Hoist runners	0.35	0.65	0.70	0.70	0.75	0.70	0.75
Locomotive crane operators	0.35	0.80	0.75	0.60	0.60	0.75	0.80
Iron workers	0.30	0.85	0.80	0.80	0.80	0.80	.....
Electricians	0.30	0.60	0.70	0.70	0.75	0.70	.....

The normal working day was 10 hours throughout the period of construction, except during the years 1919–20, when an 8-hour day was in force for unskilled labor. The working day for skilled labor varied from 8 to 10 hours, depending on the class. In 1930, the 8-hour working day was adopted for all classes of labor. In all cases, time and one-half was allowed for overtime, with double time for Sundays and recognized labor holidays.

As the duration and extent of the construction operations each year were largely governed by weather conditions, the number of men engaged in the Ship Canal work varied from month to month. It reached a minimum during January or February each year when concrete and earth operations were largely suspended, while the maximum monthly average force during the post-war period of operations ranged from 1 462 in November, 1920, to 3 728 in August, 1926.

There was a marked increase in this force from 1924, because of construction on Sections Nos. 6, 7, and 8, which were put under contract during this period. The force reached a maximum in 1926, because the work on the lower sections, Nos. 1, 2, 3, and 4, was still in full swing with the extensive opening up of construction operations on the upper sections, Nos. 6, 7, and 8.

Summary of Costs

The cost of excavation of earth and rock, and the concrete of the various structures, constitutes approximately 58% of the total of $129 954 000 (see Table 2). The description of procedure which follows will be confined largely to these three features of the work.

TABLE 2.—Distribution of Total Cost, Welland Ship Canal

Description of items	Total cost	Pre-war cost	Expenditures to December 31, 1931	
			Dollars	Percentage
Sections 1 and 2	$18 725 000	$6 672 000	$18 675 000	99.7
Sections 3 and 4	26 189 000	4 666 000	25 125 000	95.9
Section 4A	75 000	75 000	75 000	100.0
Section 4B	738 000	0	655 000	88.6
Section 5	4 868 000	1 401 000	4 829 000	99.2
Section 6	10 557 000	0	9 800 000	92.8
Section 7	10 354 000	0	10 010 000	96.6
Section 8	12 136 000	255 000	11 379 000	93.8
Cement	11 482 000	1 006 000	11 410 000	99.4
Steel gates	5 773 000	0	5 538 000	96.0
Bridges	6 850 000	208 000	6 839 000	99.8
Lock metal work and machinery	4 961 000	160 000	4 500 000	90.7
Right of way	1 926 000	853 000	1 905 000	98.9
Transmission line and equipment	1 820 000	0	1 775 000	97.5
Power house and equipment	809 000	0	490 000	60.6
Construction railway	1 914 000	260 000	1 860 000	97.2
Gate dock	450 000	0	450 000	100.0
Gate lifter	782 000	0	765 000	97.8
Buildings	543 000	0	500 000	92.1
Unwatering gates	302 000	0	302 000	100.0
Engineering	8 700 000	430 000	8 376 000	96.3
Totals	$129 954 000	$15 986 000	$125 258 000	96.4
Included in above are:				
Earth excavation	$25 232 000	$4 661 000	$24 455 000	96.9
Rock excavation	18 903 000	2 195 000	18 259 000	96.6
Concrete and cement	31 354 000	1 420 000	30 774 000	98.2

Earth Excavation

The earth removed in the construction of the Welland Ship Canal represents, approximately, 20% of the total cost of the entire undertaking, about 50 000 cu. yd. of excavation being involved.

Costs, Quantities, and Unit Prices.—Table 3 gives the range of contract unit prices for earth excavation with the quantities moved. It will be noted that the prices range from $0.1975 per cu. yd., in connection with the dredging on Section No. 1 during the pre-war period, to $4.86 per cu. yd. for tunnel excavation in connection with the gate-dock construction at Port Weller. The average is $0.4501 per cu. yd.

It is also to be noted that earth excavation prices prevailing during the pre-war period, from 1913 to 1916, were practically doubled on these same sections in the post-war period. In addition to the quantities in Table 3, handled on a unit-price basis, 2 639 000 cu. yd. were removed from Sections Nos. 1, 2, 3, 4, and 5, during the "cost plus" period from 1919 to 1922.

In the three cases (see Table 3) in which the unit price for excavation equals or exceeds $1.00 per cu. yd., this high price is due to some special operation being necessary in connection with the excavation work; as, for instance, in the case of the $1.50 price on Section No. 6, excavation was

removed from within deep steel sheet-pile enclosures. The $4.86 price at the gate-dock was for earth removed in the construction of the emptying tunnel from this structure, by tunneling methods.

Plant.— Standard types of excavating equipment were generally used throughout the construction operations. The dragline took a very prominent part and proved particularly effective in the excavation of the prism sections where flat side slopes did not lend themselves to economical trimming by shovels. The much greater reach of the dragline, both horizontally and vertically, largely eliminated the extensive track-shifting operations that shovel operations would have necessitated. In many cases shovels excavated the center core and the draglines finished along the slopes, all loading into standard-gauge dump cars.

TABLE 3.—UNIT PRICES FOR EARTH EXCAVATION

Section Nos.	Period	Index	Quantities, in cubic yards	Unit price	Total cost
1	1913–16	Wet	1 570 000	$0.1975	$310 075
1	1913–16	Dry	1 685 000	0.34	585 537
2	1913–16	Prism	3 132 000	0.33	1 033 560
2	1913–16	Lock	2 010 000	0.40	804 000
2	1913–16	Breast walls	23 000	0.75	17 250
1 and 2	1922–30	Wet	1 167 000	0.84	991 950
1 and 2	1922–31	Dry	3 853 000	0.645	2 485 185
3	1913–16	General dry	2 254 000	0.35	788 900
3	1913–16	Trench	3 000	0.60	1 800
3	1913–16	Trench, deep	6 000	0.90	5 400
3 and 4	1921–31	General	5 200 000	0.60	3 120 000
4A	1913–16	General	79 000	0.35	27 650
4B	1926–30	Wet and dry	396 000	0.40	158 400
5	1913–16	Wet and dry	3 817 000	0.275	1 049 675
5	1921–31	Wet and dry	1 058 000	0.66	698 280
6	1926–31	Prism	9 399 000	0.375	3 616 265
6	1926–30	Siphon, open	631 000	0.63	397 530
6	1926–30	Siphon, trench	183 000	1.50	274 500
6	1926–30	Culvert, wet	1 066 000	0.45	479 700
7	1926–31	Wet and dry	10 213 000	0.42	4 281 060
8	1925–30	Dry	1 037 000	0.67	694 790
8	1925–30	Wet (harbor)	415 000	0.60	249 000
8	1925–30	Wet (lake)	20 000	1.00	20 000
Gate-dock	1928–30	Tunnel	4 000	4.86	19 440
Gate-dock	1928–30	General	180 000	0.40	72 000
Totals	.....	.....	49 281 000	0.4501	$22 181 947

The construction of the entirely artificial harbor at the north end of the Ship Canal at Port Weller, in Lake Ontario, necessitated the formation of two embankments, each extending 1½ miles out into the lake from the original shore line. These embankments were made with the surplus earth and rock excavation originating on the north end of the work between Thorold and Lake Ontario.

This material was removed from the prism at a number of different points, to its final destination in the lake, by several contracting organizations.

Construction Railway.—To unify this lake disposal, the Department of Railways and Canals provided a double-track, standard-gauge railway from the foot of the Escarpment at Thorold to Lake Ontario, a distance of 7½ miles, and managed it with a complete dispatching force. Sufficient sidings were provided from the railway to the canal prism to accommodate all excavation operations under way. Upon their arrival on the main line from any of the sidings, all contractors' trains operated under train orders, the block sys-

tem of operation being used, and as long as a train remained on the main line of the construction railway its crew carried on entirely under the instructions and orders of the Railway Operating Staff.

The Lake, or Harbor, Embankments.—For the disposition of this material forming the lake embankments, 2-pile trestles were erected following approximately the center line of each embankment, and as each trestle was projected farther and farther into the lake it was immediately filled with material from the excavation. At the same time the trestle fills were widened as rapidly as the production of excavation permitted. These lake fills settled extensively from time to time and this introduced serious delays to the disposal operations; but perhaps the severest conditions were experienced during the winter months, when a combination of low temperature and heavy seas threatened at times to eliminate the embankments entirely. (See Fig. 3.)

From 1913, when the first of the excavation was placed in these embankments, until 1927, when they were completed with a facing of rock for protection, approximately 10 000 000 cu. yd. of excavated material were deposited in these construction units. All the operations involved in the excavation, transportation, and final disposal, were done under the unit prices given for Sections Nos. 1, 2, and 3. (See Table 3.)

A proportion of the necessary excavation was required for the back-filling of the structures and this was usually placed with comparatively short hauls. Some of the prism excavation was utilized in the construction of water-tight embankments along the prism where canal-water level would be above the original contour of the ground.

Water-Tight Embankments.—The construction of these water-tight embankments required the selection of the material and its proper consolidation in the embankments. This was generally effected by the use of elevating grader units loading into wagons which were drawn by mules, although in the last few years of construction the mules were replaced, to a limited extent, by gasoline tractors. However, the mules or the tractors were essential features of this operation, in that they provided a compacting medium.

Hydraulic Dredging.—On the summit level of the canal, where, to a large extent, the excavation of the Ship Canal prism represented the widening and deepening of the Third Welland Canal, excavation was carried on chiefly as a dredging operation; and as no disposal grounds for dredged material were available in the summit level itself, this introduced an expensive and long tow for the transportation of all material in scows to Lake Erie. The only available dumping ground was about two miles out in the lake, beyond the Harbor of Port Colborne. This disposal operation represented a tow from Section No. 5 to the dumping ground in Lake Erie of about sixteen miles, with the additional disadvantage of possible delay due to rough weather on the lake.

In handling this situation on Section No. 5 (the northerly section of the summit level, between Allanburg, Ont., and Port Robinson), the dredging contractor began operations with two standard type dipper dredges—the *Monarch* and the *Excelsior* (fitted with 55-ft. spuds and 5-yd. buckets). (See Table 4.) These dredges loaded into bottom-dump scows of all-steel

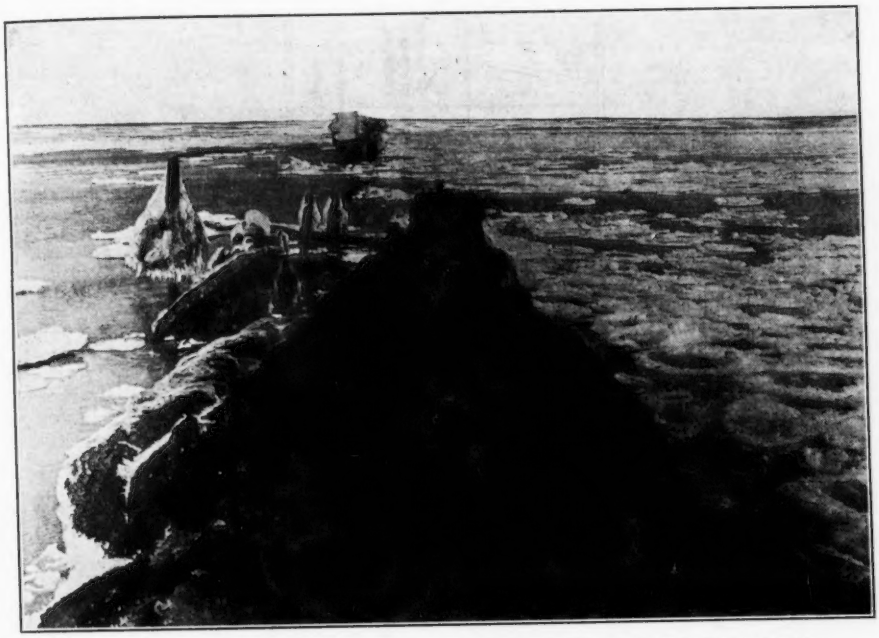

FIG. 3.—GAP BETWEEN WEST DUMP AND TRESTLE AT PORT WELLER HARBOR

FIG. 4.—A 20-INCH HYDRAULIC DREDGE IN SPECIAL PUMPING BASIN

TABLE 4.—Comparison of Dimensions; Dredges Used on Welland Ship Canal.

Name of dredge	Type	Construction	Dimensions of Hull, in Feet			Diameter of suction, in inches	Diameter of discharge, in inches	Main Pump Engine or Motor			Main Pump	
			Length	Beam	Depth			Type	Horse-power	Normal full load speed, in revolutions per minute	Outside diameter, in feet	Impeller diameter, in inches
Monarch	Dipper	All-steel	100	36	10	.....	.....	.....	.....	.....	.....	.....
Excelsior	Dipper	All-steel	100	36	10	.....	.....	.....	.....	.....	.....	.....
Primrose	Hydraulic	All-steel	136	42	11	20	20	Triple expansion	800	200	.....	.....
Stewart	Hydraulic	All-steel	100	36	11	20	20	Direct-connected variable speed, electric motor.	1 500	500	8	57
Shuniah	Hydraulic	All-steel	125	40	10	22	20	Direct-connected variable speed, electric motor.	1 500	500	8	57
Tornado	Hydraulic	All-steel	175	42	12	24	24	Triple expansion	1 800	182	10	60

construction, with capacities of 500 cu. yd. In this initial operation the excavation and loading operation cost 30½ cents per cu. yd., while the towing and disposal cost 40 cents per cu. yd., making a total cost for the dipper dredge excavation (with disposal in Lake Erie) of 70½ cents per cu. yd.

The long tow, combined with the lost time experienced at Port Colborne with the disposal operation in the lake being held up by rough weather, so reduced the output of the dredges that a more continuous method of disposal became necessary. To effect this the contractor placed a 20-in. hydraulic dredge, the *Primrose*, on the section (see Table 4).

However, as there was no territory available for use as a hydraulic disposal ground, throughout the length of the section, it was not possible for this dredge to excavate in the prism and discharge directly to the available disposal ground. Instead, the dredge was located in a basin excavated on one side of the canal prism entirely clear of the navigation channel, with the hydraulic disposal area immediately alongside. The output of the dipper dredges, excavating the prism, was brought by scow, and dumped in front of the hydraulic dredge, which rehandled it into the disposal area (see Fig. 4). By this arrangement the average tow was reduced from 16 to 2 miles, and the frequent delays due to rough weather on Lake Erie were entirely eliminated.

With the consequent vastly increased output obtained by adopting this method of disposal, and because of the marked reduction in the towing distance, the cost of transportation to the hydraulic dredge was 18 cents per cu. yd., as against 40 cents for disposal in Lake Erie. The additional rehandling cost for the dredge was 14 cents per cu. yd., making the total cost of this second method, 62½ cents per cu. yd. (exclusive of plant rental charge), using the hydraulic dredge, as against 70½ cents for lake disposal.

The foregoing methods of excavation were used on Section No. 5, both in the pre-war period and again in the post-war period, but the costs of these operations, as quoted, apply only to the post-war period of operation, when the unit price per cubic yard for earth excavation was 66 cents. Approximately 900 000 cu. yd. of earth were moved.

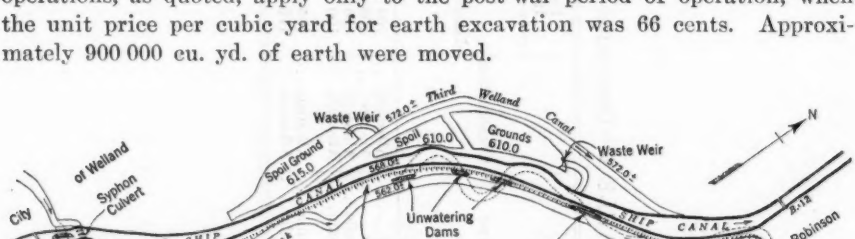

FIG. 5.—PLAN SHOWING RELOCATION OF CHIPPAWA CREEK, NEW PRISM LOCATION, SECTION 6.

Operations in Section No. 6.—The construction of Section No. 6, extending from Port Robinson to Welland, Ont., was started in the fall of 1925, and while this section also forms a portion of the summit level, it represented the construction of an entirely new prism, involving as well the relocation of Chippawa Creek. These conditions are indicated in Fig. 5.

The diversion of Chippawa Creek from the area occupied by the Ship Canal prism to a location immediately alongside, and the excavation of the canal prism, required the moving of about 10 800 000 cu. yd. of earth. Furthermore, it necessitated the construction of a water-tight embankment practically throughout the entire extent of the section to separate the canal prism from the immediately adjacent Chippawa Creek. The water level in the creek was approximately 10 ft. lower than the water level in the Third Welland Canal.

The excavation of this prism was effected by the combined use of electrically operated hydraulic dredges and draglines. In addition, the extensive embankment work involved the use of several grading units which excavated material from the prism and transferred it in wagons, drawn by mules or gasoline tractors. The material was deposited in the embankment in thin layers, and the travel of the mules or tractors provided the required consolidating medium.

Two electric draglines fitted with 175-ft. booms and 5-yd. buckets (see Fig. 6) were utilized chiefly in excavating the slopes of the prism above water level. Their output was sidecast into the middle of the prism for final disposal by the hydraulic dredges. This procedure, although involving the rehandling of a large quantity of material, was found more economical than hauling the material by train to the final point of disposal.

The hydraulic excavation of the prism presented several interesting features. The two hydraulic dredges used—the *Stewart* and the *Shuniah* (see Table 4)—were operated entirely by electricity. The main pump, cutter, ladder, and spud hoists, as well as all general service pumps, were motor-driven.

The pontoon and land discharge lines were of 20-in. welded steel pipe. Besides the equipment described in Table 4, an independent motor of 150-h. p. capacity ran the cutter, while a 60-h. p. motor operated the ladder and spud hoist. All these motors worked on a frequency of 25 cycles and a pressure of 2 200 volts.

One of the first operations necessary on Section No. 6 was the excavation of the new channel for Chippawa Creek, in order to divert this stream from the Ship Canal prism. Accordingly, the two dredges, were brought up Chippawa Creek from the Niagara River. The hydraulic spoil areas were formed immediately adjacent to the prism by draglines casting up an earth embankment around the perimeter of each disposal area. A timber waste weir was introduced at some point in this embankment, of such construction that by the use of stop-logs the waste water was allowed to drain from the disposal area back to the creek, after the desired settlement had occurred.

In the excavation of the creek diversions the dredges pumped the material an average distance of about 1 500 ft. against an average reverse head of 50 ft. The tops of the disposal-area embankments were generally at a height of approximately 50 ft. above the water level in Chippawa Creek.

With the completion of the channel diversions of the creek, the portions of the original creek bed in the prism area were isolated from the new creek

channel by earth and rock dams, and after these isolated areas were unwatered, the construction of the water-tight embankment separating the prism from the creek was carried on by the grading units as previously described (see Fig. 5). However, unsatisfactory progress with the construction of this dividing embankment caused the contractor to adopt special procedure in order to maintain the hydraulic dredges in operation.

It was impossible to connect the new prism of the Ship Canal to the prism of the Third Welland Canal at either the north or south ends, until the water-tight embankment between the new prism and Chippawa Creek was entirely completed to its full height. Therefore, the dredge, *Stewart,* was placed in the flooded portion of the new prism area. The unwatering dam was then completed and this latter operation isolated the dredge entirely, both from Chippawa Creek and the Third Canal.

To permit the construction of the water-tight embankment along this portion of the prism, inside the unwatering dam, the dredge excavated a small area in the prism, at her isolated position, to a depth 12 ft. below Ship Canal grade. Then, with the unwatering dam completed between the Ship Canal prism and Chippawa Creek, the dredge itself unwatered all this area, except the part excavated below prism grade, where it remained afloat and idle while the construction of the adjacent water-tight embankment, at this point, was continued.

When this embankment was built to 15 ft. above Ship Canal grade, the area of the prism occupied by the dredge was again flooded by pumping water from the creek over the partly completed embankment into the prism area. With this area flooded to a depth of 12 ft. above Ship Canal grade to give the dredge flotation, the excavation operation was resumed. This stage is very well indicated by Fig. 7.

At the beginning of this phase of the hydraulic operation, the dredge operated on a discharge line about 1 000 ft. long and against a reverse head of 60 ft.—the difference in level between the initial water level in the partly flooded prism and the top of the embankment around the hydraulic disposal area in use. As the construction of the adjacent water-tight embankment of the prism progressed, the contractor was permitted to raise the level in which the dredge operated, thus reducing the reverse head on his discharge.

In this operation, the waste water from the disposal area was returned immediately to the area of the prism in which the dredge operated. Any water lost or additional water required to raise the level, was supplied, as before described, by pumping from the adjacent Chippawa Creek.

In the summer of 1929 the dividing embankment between the new prism and the Creek was completed to its full height, and in September of that year the *Stewart* cut through from the new Ship Canal prism to the Third Canal, at Port Robinson.

The *Shuniah* was not isolated in the Ship Canal prism like the *Stewart,* but was utilized, instead, in the excavation of that portion of the new prism at the south end of Section No. 6 which represents largely an operation of widening and deepening the Third Canal prism. (See Fig. 5.) In this operation, a hard gravelly clay was encountered, and as the prism excavation

FIG. 6.—ELECTRIC DRAGLINE EXCAVATING THE SLOPES.

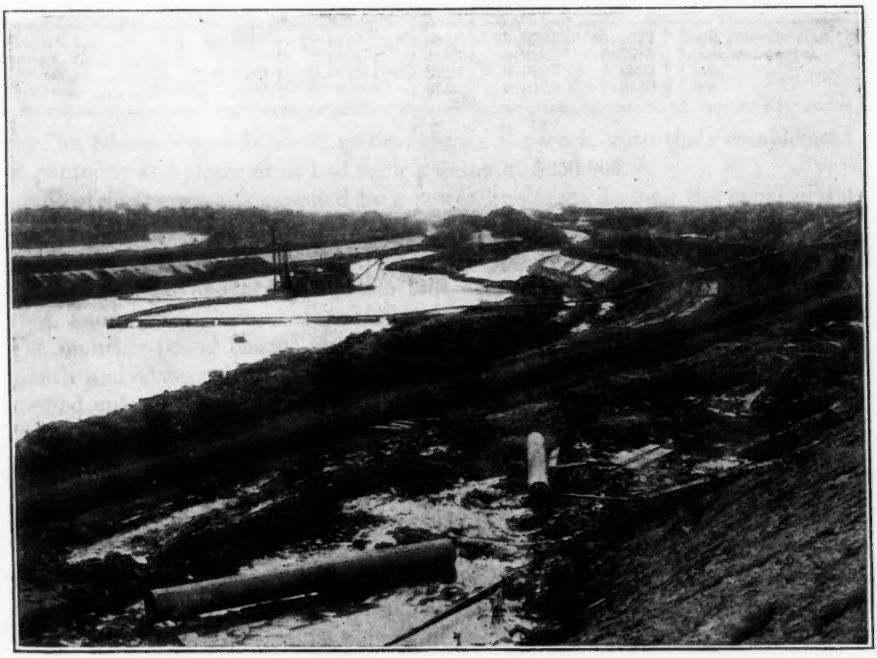

FIG. 7.—HYDRAULIC DREDGE IN FLOODED CANAL PRISM; RIVER BEYOND.

extended 20 ft., or more, above the water level in which the dredge worked, there were serious difficulties and delays. The hard character of the material and its tendency to break off in large masses, when slightly undermined by the dredge cutter caused slides which did serious damage to the dredge, and thus increased both operating and repair costs.

To meet this condition a hydraulic "Giant" throwing a $1\frac{1}{2}$-in. stream, under a pressure of 125 lb. per sq. in., was mounted on the forward end of the dredge. It played constantly on the face of the excavation above water level during the operation of the dredge. This arrangement effectively eliminated the difficulty previously experienced from slides by bringing down the material above water level so that no overhanging face existed at any time in front of the dredge.

In these hydraulic operations the two dredges, the *Stewart* and the *Shuniah*, operated three 8-hour shifts daily, closing down only on Sundays and recognized labor holidays. Cold weather, with its accompanying ice conditions, generally necessitated the dredges being laid up during January, February, and half of March of each year.

Data with respect to the operating cost of the two hydraulic dredges will be of interest at this point. The costs in Table 5 do not include overhead and plant rental; they represent actual operating costs only.

TABLE 5.—COMPARISON OF HYDRAULIC DREDGE OPERATING COSTS PER CUBIC YARD

Dredge	Wages	Power	Supplies	Repairs	Total	Material
Shuniah	$0.0472	$0.0706	$0.0957	$0.0294	$0.2429	Hard gravelly clay.
Stewart	0.0339	0.0538	0.0562	0.0139	0.1578	Loam and gravel.
Tornado	0.0640	0.0623	0.0258	0.0299	0.1820	Clay.
Primrose	0.0900	0.0508	0.0550	0.0172	0.2130	Clay.

The *Shuniah* and *Stewart*, as brought on the work, with their complement of pontoons and shore pipe, had each a value of $250 000.

Electric energy was supplied by a power line erected along the canal prism from one of the local distribution systems of the Hydro-Electric Power Commission of Ontario. From this feeder line the power was transferred to the dredge by a flexible armored cable laid over land to the edge of the water area and carried across the water on the pontoon line to the dredge. The monthly power charge was based on a 10-min., peak-load reading for the month and averaged $4 000 per month for each dredge. This power charge applied only during the months that the dredge was actually connected to the Commission's distribution system.

The output of these hydraulic dredges, and the actual number of hours worked during the three seasons, 1926, 1927, and 1928, are listed in Table 6. From this information the average hourly output of each dredge is deduced.

It will be noted that the dredge, *Shuniah*, operating in hard gravelly clay, obtained a higher hourly output than the *Stewart*. This is due to the fact that the latter had to pump against a constant reverse head of 60 ft., while the *Shuniah* operated against a head of 40 ft. and less during the years 1927 and 1928.

Earth Excavation in Section No. 7.—This section extended from Welland to a point about 1.5 miles north of Port Colborne. The widening and deepening of the Third Canal prism to Ship Canal dimensions on this part of the work involved the removal of approximately 9 800 000 cu. yd. This operation

TABLE 6.—COMPARISON OF OUTPUT DURING THE SEASONS 1926, 1927, AND 1928

Dredge	Output, in cubic yards	Hours operated	Output per hour, in cubic yards
Shuniah	2 036 565	11 369.5	179
Stewart	1 645 132	10 590.0	155
Tornado	3 866 382	9 734.0	397
Primrose	1 231 433	8 874.0	139

was accomplished very largely by the use of two steam-driven hydraulic dredges, the *Primrose* and *Tornado* (see Table 4), assisted by two draglines, with 80-ft. booms and using 2.5-yd. buckets. The latter excavated the material on the new west slope of the Ship Canal prism above water level and sidecast it into the dredging area. As the east slope of the Ship Canal prism corresponded with the east slope of the Third Canal no similar excavation, above water level, was necessary on that side.

Hydraulic disposal areas placed as conveniently as possible to the west side of the prism were outlined by earth embankments and utilized for disposal in the same manner as described for Section No. 6. These areas were so located along the extent of the section that at no time did either of the hydraulic dredges operate on a pipe line more than 6 500 ft. in length. The average height of the disposal area above the water level in which the dredges operated was 25 ft., because the general level of the land adjacent to this section of the work was more uniform and lower than that of the ground available for spoil areas on Section No. 6.

As the main pump of the *Primrose* was under-powered, this dredge was used in digging the first lift of excavation from the prism, and it carried these cuts to an average depth of about 14 ft. below water level.

The *Tornado* followed the *Primrose* by carrying the excavation operations to the final Ship Canal grade, 26½ ft. below water level. In both cases the dredges operated with a floating pontoon line not longer than 1 000 ft. The shore line was parallel with the side of the cut and was equipped with connections for attaching the pontoon line at the required intervals. This arrangement was necessary in order to keep the slack part of the pontoon line clear at all times of that portion of the canal prism used by navigation. These dredges operated three 8-hour shifts per day, being shut down only for 24 hours on Sundays and labor holidays.

The operating costs of the dredges, *Tornado* and *Primrose,* exclusive of overhead and plant rental, were as shown in Table 5. The dredge, *Tornado,* delivered on the work, with its complement of pontoons and discharge line, was valued at $350 000, while the *Primrose,* similarly equipped, was valued at $225 000.

A matter of equal interest in connection with these two machines (see Table 6) is the output obtained, and the actual number of hours operated,

during three seasons, from which is deduced the average output per hour. It will be noted that the hourly yardage for the *Tornado* was very much in excess of that maintained by the *Primrose*. This was due partly to the fact that the *Tornado* was a larger machine, using 24-in. equipment as against 20-in. equipment on the *Primrose*. However, a greater contributing cause was the lack of power on the main pump of the *Primrose* (see Table 4).

Rock Excavation

As in the case of earth excavation, the rock was generally removed by shovels loading into standard-gauge equipment in the dry sections, and dipper dredges loading into scows in the submarine portions of the work.

Contract Prices and Quantities.—At only three points in the extent of the Ship Canal was rock found in very large quantities although, as shown in Table 7, some rock excavation was included in every section of the work. A considerable range in unit prices also prevailed, and, again, it is to be noticed that rock excavation prices in the post-war period were more than double those prevailing in the contracts awarded prior to the war.

TABLE 7.—CONTRACT PRICES FOR ROCK EXCAVATION

Section	Period	Index	Quantities, in cubic yards	Unit price	Cost
No. 1	1913–16	Wet	86 000	$4.05	$348 300
No. 1	1913–16	Dry	57 000	1.05	59 850
No. 2	1913–16	General	19 000	1.40	26 600
Nos. 1 and 2	1922–31	Wet	57 000	10.00	570 000
Nos. 1 and 2	1922–30	Dry	90 000	2.75	247 500
No. 3	1913–16	General	1 295 000	1.10	1 424 500
Nos. 3 and 4	1922–31	General	1 832 000	1.56	2 857 920
No. 4A	1913–16	General	1 000	2.00	2 000
No. 4B	1926–31	Wet and dry	190 000	1.90	361 000
No. 5	1913–16	Wet and dry	29 000	3.45	100 050
No. 5	1922–31	Wet and dry	88 000	6.25	550 000
No. 6	1926–31	General	57 000	3.15	179 550
No. 7	1925–31	Wet and dry	1 548 000	1.90	2 941 200
No. 8	1924–31	Dry	2 107 000	1.50	3 160 500
No. 8	1924–31	Wet (harbor)	679 000	3.88	2 634 520
No. 8	1924–31	Wet (lake)	165 000	4.90	808 500
Total	.....	.....	8 300 000	$1.96	$16 271 990

In addition to the quantity covered by contracts, about 720 000 cu. yd. of rock were removed from Sections Nos. 2, 3, and 5, during the "cost plus" period, making the total rock excavation quantity 9 020 000 cu. yd. removed at the end of 1931.

The major rock excavation operations occurred in the construction of Sections Nos. 3, 4, 7, and 8, and some details of these three larger operations follow.

Rock Excavation in Sections Nos. 3 and 4.—The approach of the Ship Canal on an alignment normal to the Niagara Escarpment necessitated extensive rock cutting for the accommodation of the flight lock structure at this point. The average width of this cut was 300 ft. and the depth varied from 20 to 100 ft. In addition, south of the flight lock structure, Lock No. 7 was also located in rock cutting, as well as a portion of the prism of Section No. 4 south of Lock No. 7.

This situation necessitated, in all, the removal of about 3 000 000 cu. yd. of rock and a considerable portion of it was a good grade of limestone and sandstone. The contractor on Section No. 3 was required to select suitable grades of rock from the excavation and pass it through a crushing plant, thus reducing it to the sizes required for the concrete aggregate on the work. This crushed stone was provided free of cost to the contractors on Sections Nos. 1 and 2, while on Section No. 3 the contractor was allowed to use any surplus available for his own concrete purposes without compensation from the Department for the crushing.

Accordingly, the crushing plant was located on the lower plateau, a short distance north of the foot of the flight locks. All suitable rock from the excavation was sent there immediately, while the unsuitable shaly rock was utilized for local back-filling. Standard 60 to 80-ton railway shovels were used in this operation, loading into 6 and 12-cu. yd. side-dump cars. The 6-yd. cars were used most, because of their convenience in dumping at the primary crusher of the crushing plant.

It was desired to reduce the over-break to a minimum, in order not only to reduce the rock excavation quantity, but in the same way to keep down the quantity of concrete in the structures. Therefore, rock channelers were used in Section No. 3 in the pre-war period. The channel cut was put down on the neat alignment of the back of the lock-wall, in 6-ft. lifts. However, as the rock was extremely variable in its characteristics, with alternate strata of hard limestone, or sandstone and soft shale, the channeling operation proved extremely slow and expensive. In resuming work on this section after the war, close drilling was adopted as an alternative to the previous operation of channeling.

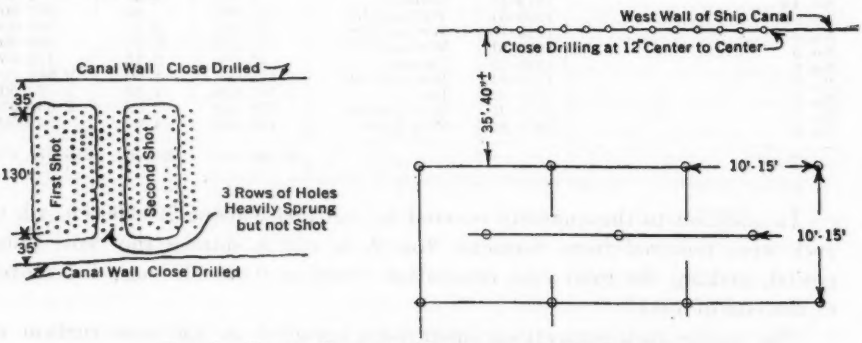

FIG. 8.—PLANS SHOWING METHOD OF CLOSE DRILLING.

In the close-drilling operation a row of vertical holes was drilled on the alignment desired for the limits of the rock cutting, these holes being drilled at intervals of about 12 in. to whatever depth the excavation was being carried (see Fig. 8). In blasting none of the blasting medium was placed in the close-drilled holes. When any part of the rock, so outlined by close drilling, was shot, the effect was to break it only to the line of the close-drilled holes, leaving a very uniform solid face exposed as illustrated subsequently under "Operations in Section No. 8."

Operations in Section No. 7.—Generally throughout the extent of the summit level, no rock was encountered above the grade of the canal prism from Allanburg south to Humberstone, at the south end of Section No. 7, but between this latter point and the harbor at Port Colborne practically all the necessary excavation for the canal and its structures was in rock.

To eliminate two very sharp bends of the Third Canal alignment as it approached Port Colborne, the Ship Canal location left the Third Canal entirely, and for the last two miles north of Port Colborne followed a perfectly straight line which involved an angular crossing of the Third Canal. (See Fig. 9.) This alignment placed about 3 000 ft. of the new prism at the south

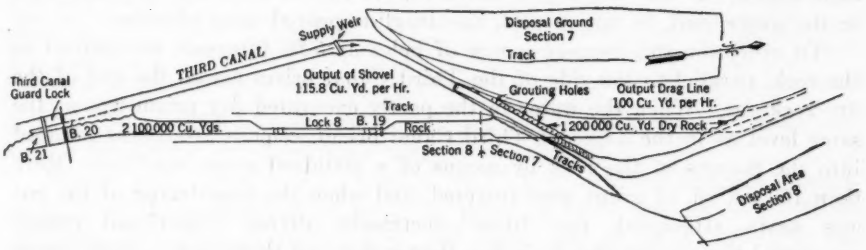

FIG. 9.—PLANS SHOWING TWO BENDS (REMOVED) IN OLD CANAL.

end of Section No. 7 entirely clear of the present Welland Canal north of the point of crossing. South of this point of crossing on Section No. 8 the remainder of the new prism, including the pit for the guard-lock, No. 8, for a distance of 8 000 ft., was located entirely clear of the Third Canal and so was made available for excavation "in the dry."

In the first section of the new prism north of the canal intersections, where about 1 200 000 cu. yd. of rock were available for dry excavation, the contractor carried on all his drilling for shooting and close drilling, using air-driven stand drills fitted with $1\frac{1}{8}$-in. hollow, drill steel and drilled the full depth of the cut which varied from 35 to 40 ft. in one lift.

This drilling was started first with the holes spaced 5 ft. apart across the prism of these rows, at intervals of 15 ft.; but as this gave unsatisfactory results from the shooting, the spacing of holes was changed to 8-ft. squares and much more uniform break-up resulted. At this point in the Ship Canal prism again the use of close drilling was required in order to economize in excavation and concrete, and lines of close-drilled holes, spaced at 12-in. intervals, were sunk to the full depth of the excavation on the back line of the concrete walls facing this part of the prism.

No holes for shooting were drilled closer than the depth of the cutting from the line of this close drilling (35 to 40 ft. (see Fig. 8)); and in loading for shooting, no dynamite was placed in the close drilled holes, but by springing and shooting the holes in the center of the prism with heavy charges of 70% and 40% dynamite all the rock was well broken up. In no case did the effect of the shooting extend beyond the line of the close drilled holes on either side of the prism.

The most interesting features of this rock operation were the methods and equipment used for excavation and disposal. The contractor employed a dragline with a 100-ft. boom and a 6-yd. bucket. Although it was rather unusual for this class of work, it proved effective, and, at the outset, had a distinct advantage over the usual shovel operation.

Much seepage occurred through the seams of stratified limestone at the south end of the area, immediately alongside of the Third Canal prism. Although the contractor placed two 12-in. pumps, together with the one 6-in. and one 8-in. centrifugal pumps in use, to unwater this small initial pit excavation, seepage considerably in excess of this pumping capacity was experienced. It was necessary, therefore, for him to excavate for some time in the water, and, in this respect, the dragline proved most effective.

To overcome this seepage a row of holes at 5-ft. intervals was drilled in the rock, parallel to the side of the Third Canal prism across the end of the dry rock cut. When the water in the partly excavated dry prism was at the same level as in the adjacent Third Canal prism, cement grout was pumped into the fissures of the rock by means of a standard grout machine. More than 100 cu. yd. of grout were injected, and when the unwatering of the cut was again attempted, two 10-in. electrically driven centrifugal pumps unwatered the cut completely in less than a day and thereafter a single pump operation kept the cut clear of water as the excavation proceeded.

The average haul from the point of loading to the disposal ground was about 7 500 ft. Two trains, each with one locomotive and three 30-yd. cars, were able to handle the full output of the dragline, making the round trip to and from the dump in 15 min. In September, 1925, this machine removed 80 500 cu. yd. of rock from the prism, and its average monthly output, representing an average loading rate of 100 cu. yd. per hour of actual working time, was 45 200 cu. yd. The dragline was working a continuous 24-hour shift when this maximum monthly output was obtained.

The unit operating costs were, as follows:

Item	Dragline excavator	Six-yard shovel
Wages	$0.0775	$0.1223
Power	0.0341	.....
Supplies	0.0212	0.0644
Repairs	0.0070	0.1395
Total	0.1398	0.3262
Material	Rock	Rock; Section No. 8

The 1¾-in. drag cable was renewed every 10 days and the 1-in. hoist cable every 30 days.

The dragline was operated entirely by electricity, the main motor having a rating of 225 h.p., the hoist motor, 175 h.p., and the swinging motor, 75 h.p., all operating on 60-cycle, 2 200-volt energy.

The cost of rock excavation, in Section No. 7, in dollars per cubic yard, exclusive of overhead and plant rental charges, was as listed in Table 8.

The value of the plant and equipment installed for this operation was $300 000. A centrally located, 2 200-cu. ft. compressor, operated by elec-

tricity, delivered air to the drills at 100 lb. pressure. The average rate of drilling was 22 ft. per hour, based on actual drilling time. The average depth of holes was 38 ft. and holes were completed with one set-up of the drills.

Each cubic yard of rock removed required 8 in. of drilling, including close drilling. The dynamite required for each cubic yard was 1.5 lb. Shots

TABLE 8.—UNIT COSTS OF ROCK EXCAVATION PER CUBIC YARD

Operation	Section No. 7	Section No. 8
Drilling	$0.1592	$0.1290
Blasting	0.4724	0.2870
Excavating and loading	0.1398	0.3550
Transportation	0.1723	0.2460
Disposal	0.0863	0.1530
Field operating cost	$1.03	$1.1700

were limited to about 12 000 cu. yd. of rock each, to minimize the movement of the dragline clear of the area being shot.

For all rock excavation on Section No. 7, both dry and wet, the Department paid the contractor at the unit price of $1.90 per cu. yd. About 378 000 cu. yd. of rock at the north and south ends of the dry work, at the intersections of the two canal prisms, required removal by dredging.

Operations in Section No. 8.—The dry rock excavation on Section No. 8, which included the removal of about 2 100 000 cu. yd., was in many respects similar to the operation previously described, but by comparison it had several interesting features.

The rock in this section was a stratified limestone formation typified by heavy beds interspaced with many seams of soft shale. The depth of the cutting averaged 35 ft., with the width of the excavation varying from a minimum of 90 ft. in the lock area to a minimum of 220 ft. in the prism above and below the lock.

On account of a heavy ledge of rock that occurred throughout the excavation approximately midway in the depth of the cutting, and in order to expedite the work by avoiding the use of extremely long steel, the drilling was done in two lifts of about equal depth. It was done with drills of the anvil block type equipped with independent air motors for rotating the drill steel. Drills were mounted on frames, each about 15 ft. high, on which they were raised and lowered by means of a small winch. These machines used hollow drill steel $1\frac{1}{4}$ in. in diameter, with starting bits $2\frac{7}{8}$ in. across the wings, the bits of the longer drills being reduced by $\frac{1}{8}$ in. with each increase in length.

At the outset the holes were spaced at the corners of 15-ft. squares. No holes were drilled for shooting within 35 ft. of the limit of the excavation line, where close drilled holes of 12-in. spacing were put down to the full depth of the cut (see Fig. 10). After these holes were shot the excavation revealed that the rock had broken very unsatisfactorily. Still retaining the 15-ft. spacing of holes an additional hole was drilled for about one-half the depth in the center of each 15-ft. square, with the idea that with this hole loaded midway of the depth of the cut the break-up would be better (see

Fig. 8). This improvement, however, did not materialize and as a final procedure the spacing of holes was reduced to 10-ft. squares, all sunk to the same depth. This proved satisfactory although in the progress of the excavation a considerable amount of jack-hammer work was required to break up large pieces of the rock slabs not effectively broken by the original shot (see Fig. 11).

From records of the drilling done in this rock operation the following averages are of interest:

Feet drilled per hour for blasting.....	15.3
Feet per hour, close drilling	20.5
All drilling, in feet, including jack-hammers.....	17.6

At the beginning of the drilling operations in Section No. 8, tripod drills with a screw feed of 30 in. were used, and they continued in use after the introduction of the stand drills. It is of interest to note from the drilling records of this work that the stand drill, both on the drilling for blasting, but more particularly on the close drilling work, obtained an average hourly output almost 50% in excess of the tripod drill. This was due to ease of shifting from hole to hole and the greater rapidity obtained in changing steels.

The maximum feet drilled per hour, as indicated by the drill records, was obtained in April, 1925, when the stand drills, engaged on close drilling the sides of the rock cut, obtained an average of 27.8 ft. drilled per hour.

When a satisfactory 10-ft. spacing had been developed, the excavation area was drilled and divided into shots comprising 80 to 90 holes per shot. Before final loading, each hole was independently sprung four times, the first spring using two sticks, and the fourth or last spring, fifty-five sticks, of 60% dynamite.

As the shooting of a group of holes generally spread the rock a distance of about 30 ft. beyond the limit of the area shot, the next three lines of holes ahead of that area were always sprung as heavily as possible without interfering with the loading of the holes. In this way, the rock in this adjacent 30-ft. strip was sufficiently broken up with the main shot that the actual loading and shooting of these three rows of holes never proved necessary. (See Fig. 8.) No hard rock was encountered between areas actually blasted.

In preparing an area for blasting, 80 to 100 holes were loaded in the bottom with about 100 lb. of 40% dynamite. With two caps in each hole a shot was fired by the use of a current from an adjacent 220-volt circuit. Each of these shots move from 8 000 to 10 000 cu. yd. of rock. In this rock operation, 1.1 lb. of dynamite were used per cu. yd. of rock, in the combined springing and blasting operation.

Although the rock was of a variable nature, the excavation in all cases left a uniformly straight face on the line of the close drilling, with about one-half of each hole still showing in the exposed face (see Fig. 12). Sometimes a hard toe would be found along the bottom of the cut, and this was removed by loading what remained of the close-drilled holes with a light charge of dynamite. This shooting in no way damaged the close-drilled face.

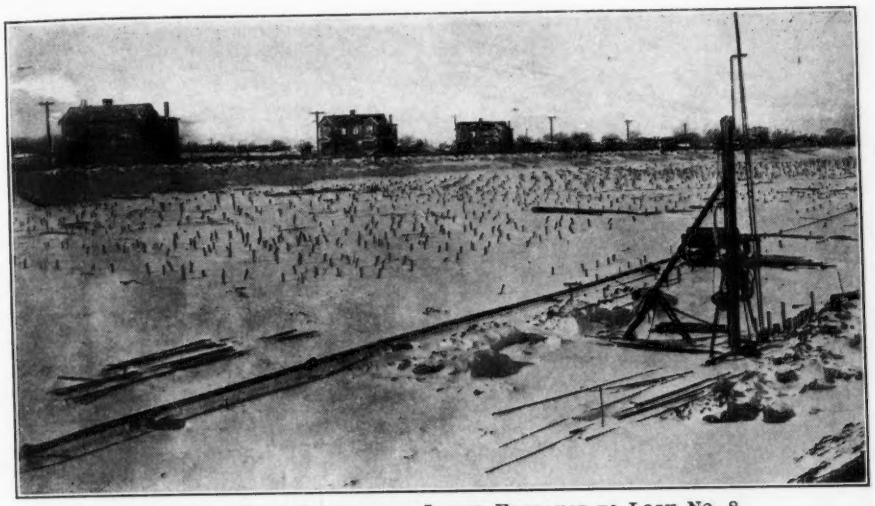

FIG. 10.—ROCK DRILLING IN LOWER ENTRANCE TO LOCK NO. 8.

FIG. 11.—EXAMPLES OF LIMESTONE ROCK SLABS TO BE BROKEN BY JACK-HAMMER.

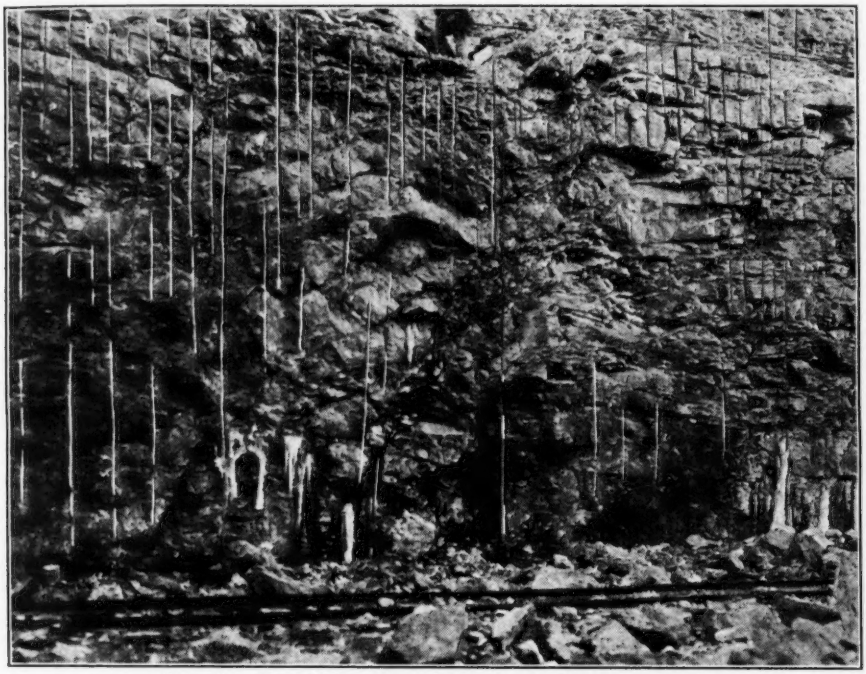

FIG. 12.—EVEN ROCK FACE SECURED BY CLOSE DRILLING.

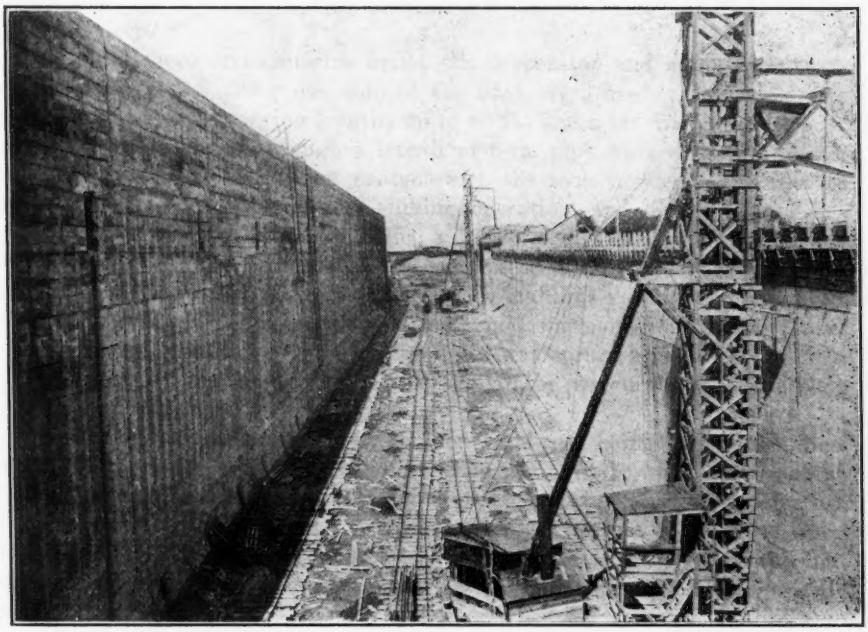

FIG. 13.—VIEW OF ARRANGEMENT OF CONCRETE PLANT FOR THE CONSTRUCTION OF LOCK NO. 1.

This rock was removed in two lifts, the first extending down approximately one-half the total depth of the cutting. Two standard railway shovels with bucket capacities of 3½ and 2½ yd., assisted by a small revolving shovel with a 1¼-yd. bucket, were used in the removal of this first lift. The small machine cleaned up behind the larger shovels in order to allow the drilling of the second lift to follow closely behind the first lift excavation.

For the excavation of the second lift to final grade, a revolving shovel, with a 6-yd. dipper and an 85-ft. boom, was used. It could load into cars 68 ft. above the level of its own tracks and could clear a 90-ft. width at grade. The machine was mounted on four standard-gauge trucks, one in each corner of the base, and was self-propelled.

In Table 8, the combined excavating and loading operations of the 6-yd., 3½-yd., 2½-yd., and 1¼-yd. shovels cost 35½ cents per cu. yd. The total value of the plant and equipment on the rock work in Section No. 8 was $500 000. For the dry rock excavation on this section, the Department paid the contractor at the unit price of $1.50 per cu. yd.

In Port Colborne Harbor the work done in connection with the Welland Ship Canal consisted of approximately doubling the harbor width and deepening the entire harbor from an existing 20-ft. draft to an ultimate 30-ft. draft at extreme low-water level of Lake Erie, involving the removal of about 844 000 cu. yd. of rock, as well as about 300 000 cu. yd. of earth overlying the rock. This work was started in 1925 being carried on by two dipper dredges equipped with 6-yd. and 4-yd. buckets, respectively, and was completed in 1931, except the final cleaning up and sweeping. In drilling the submarine rock, three boats were used, each being 80 ft. long by 30 ft. beam, and 8 ft. deep.

Standard types of submarine drills, steam-operated and mounted in carriages which travel along one side of the boat, were used. Drill steel was 2 in. in diameter, in varying lengths up to 40 ft., and a bit diameter of 3¾ in.

Each drill operated through a length of 6-in. pipe worked down through the overlying earth to a direct contact with the rock surface. In order to get rid of the earth in this pipe-sinking operation, and also to keep holes clear of the rock chips from the drills, a ½-in pipe was carried to the bottom of the hole with the drill steel and compressed air at 100 lb. per sq. in. forced the earth and rock chips up the 6-in. pipe, thus keeping the hole clear throughout the drilling operation. As each hole was completed, it was immediately loaded and shot. None of the holes was sprung; with spacing 5 ft. apart both ways and by using 1¼ lb. of dynamite per cu. yd., a well-broken rock was obtained.

Each drill averaged about 7 ft. per hour, the rock being very tough limestone. This drilling and blasting cost $1.50 per cu. yd., while the dredging and disposal, exclusive of overhead and plant rental, cost $0.75 per cu. yd. The unit price paid by the Department was $3.88 per cu. yd. in the harbor, inside the breakwater, and $4.90 per cu yd. for rock removed in the lake channel beyond the harbor breakwater. The plant in use, including drill boats, dredges, tugs, and scows, represented a total cost on the work of $650 000.

Concrete

A variety of uses was made of concrete in the construction of the Welland Ship Canal. Not only were the principal structures, such as the locks, regulating weirs, and waste weirs, of concrete construction throughout, but all bridge substructures, whether built "in the dry" or by submarine operations, dock walls, harbor and breakwater cribs, and the protection of the prism slopes, were entirely of concrete construction. This range of application naturally resulted in a corresponding range of unit prices being obtained in placing all the work under contract, as indicated in Table 9.

TABLE 9.—CONTRACT PRICES FOR CONCRETE

(All exclusive of cement)

Section Nos.	Period	Index	Quantities, in cubic yards	Unit price per cubic yard	Amount
1	1913–16	Mass concrete	168 000	$2.55	$428 400
1	1913–16	Harbor cribs	18 000	3.05	54 900
1	1913–16	Retaining walls	8 500	2.85	24 225
1	1913–16	Mass concrete	1 000	3.95	3 950
1	1913–16	Reinforced concrete	9 500	4.65	44 175
2	1913–16	Mass concrete	182 000	2.57	467 740
2	1913–16	Bridges	1 500	5.30	7 950
2	1913–16	Bridge walls	20 000	5.40	108 000
2	1913–16	Mass concrete	1 000	4.70	4 700
2	1913–16	Reinforced concrete	1 000	8.50	8 500
1 and 2	1922–31	Mass concrete	487 000	5.25	2 556 750
1 and 2	1922–30	Harbor cribs	29 500	7.00	206 500
1 and 2	1922–30	Bridges	52 500	12.00	630 000
1 and 2	1922–30	Protection	89 000	7.50	667 500
3	1913–16	Mass concrete	47 000	2.65	124 550
3	1913–16	Bridges	21 000	6.00	126 000
3 and 4	1921–30	Mass concrete	1 321 000	4.14	5 468 940
3 and 4	1921–31	Bridges	103 500	9.10	941 850
3 and 4	1921–30	Protection	35 500	7.87	279 385
4A	1913–16	Mass concrete	3 000	5.25	15 750
4B	1926–30	Mass concrete	4 000	10.00	40 000
4B	1926–30	Protection	2 200	7.50	16 500
5	1913–16	Protection	1 500	5.00	7 500
5	1922–30	Mass concrete	9 000	10.00	90 000
5	1922–30	Protection	1 000	11.00	11 000
6	1926–30	Mass concrete	11 500	8.00	92 000
6	1926–30	Siphon culvert	97 500	7.50	731 250
6	1926–30	Bridges	2 000	15.25	3 050
6	1926–30	Protection	26 500	7.50	198 750
7	1925–30	Mass concrete	112 000	8.00	896 000
7	1925–30	Protection	17 000	7.00	119 000
8	1924–30	Mass concrete	340 000	4.70	1 598 000
8	1924–30	Bridge cribs	37 000	15.00	555 000
8	1924–30	Bridges	10 000	11.00	110 000
8	1924–30	Road concrete	1 500	8.00	12 000
Gate-dock	1928–30	Mass concrete	38 000	5.95	226 100
Gate-dock	1928–30	Tunnel concrete	1 500	15.50	23 250
Totals			3 311 700	$5.10	$16 899 165

Besides the concrete placed under the various contracts, an additional 247 000 cu. yd. were placed during the cost plus period, making the total concrete placed in the Welland Ship Canal, 3 558 700 cu. yd. at the end of December, 1931.

The usual 100% or more increase in prices from pre-war to post-war periods is immediately apparent. The post-war prices varied from $4.14 for heavy mass concrete work, as in the flight-lock structure, to $15.50 for smaller quantities entering into the construction of the light section, heavily rein-

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forced bridges, cribs, and tunnel construction. In all cases the cement was supplied by the Department and delivered in cars on the commercial railway sidings nearest the work. Contracts were made with the cement companies by the Department for each year's total cement requirements.

All concrete operations on the Welland Ship Canal were conducted in accordance with one standard of procedure. The cement was supplied in cloth bags of 87½ lb. net weight, to the contractors, being subjected, before acceptance and delivery, to the standard tests prescribed by the Canadian Engineering Standards Association.

The concrete aggregate—stone, sand, or graded gravel—was supplied by the contractor under the unit price per cubic yard for concrete, except in the case of Sections Nos. 1 and 2, where the stone was furnished by the Department to the contractor, at the Section No. 3 crushing plant.

Testing.—All operations of mixing and placing the concrete were closely supervised by a competent staff of inspectors allocated to the various concrete plants. In addition, a special department, equipped with a complete laboratory, was maintained to assist in the control of the concrete operations. It functioned as a very essential unit of the entire concreting organization. In addition to carrying on the routine cement sampling and testing, the men in this department took every day from each plant, a 6 by 12-in., or an 8 by 16-in. cylinder of the concrete as it was being poured, and from these, after curing for a period of 28 days, they determined the compressive strength of the concrete as produced.

These tests guided and confirmed the judgment of the Field Inspection Staff in proportioning the mixtures to meet the strength required in the various classes of concrete. In the heavy wall sections, using the proportions of 1:2.5:5, an average compressive strength of 2 000 lb. per sq. in. was obtained, while in lighter sections, in the reinforced concrete work, or where water-tightness was an essential feature, the proportions of 1:2:4 gave a compressive strength of 3 000 lb. per sq. in., or better.

Plant Layout.—In the design of his concrete plants, the contractor was required to provide such measuring devices as would permit the accurate determination of proportions for the mix.

A typical single-lock structure with its regulating and waste weirs represented a quantity of about 360 000 cu. yd. of concrete to be placed in a distance of only about 1 500 ft. This condition justified the general use of central mixing plants which system was adopted. Distribution was effected by traveling hoist towers, derricks, and chuting systems.

At Lock No. 1 the concrete from the central plant at the north end was transferred by rail to traveling hoist towers, by which it was elevated, and with short chutes, placed in the forms. (See Fig. 13.) At Lock No. 2 a similar central plant delivered the concrete into 4-yd. buckets which were transferred on cars to traveling derricks, in the lock chamber. The derricks, in turn, hoisted the buckets to place. At Lock No. 3 the procedure was similar to that at Lock No. 2, except that every effective use was made of a large 120-ft. boom drag-line which as a crane handled the concrete to place.

At Lock No. 8 all concrete was placed by standard locomotive cranes with 60-ft. booms, the walls of the structure being only about 46 ft. high above chamber-floor level.

A much more extensive and elaborate concrete plant was used for the flight lock structure on Section No. 3, at Thorold. Twin Locks Nos. 4, 5, and 6 in flight, required the placing of 1 070 000 cu. yd. of concrete in the three continuous walls which constituted this structure, all confined to a distance of 3 035 ft., exclusive of entrance walls (see Fig. 14).

The twin lock chambers of standard dimensions are separated by a center wall, 60 ft. thick and 85.9 ft. high above the chamber floor level. The lower

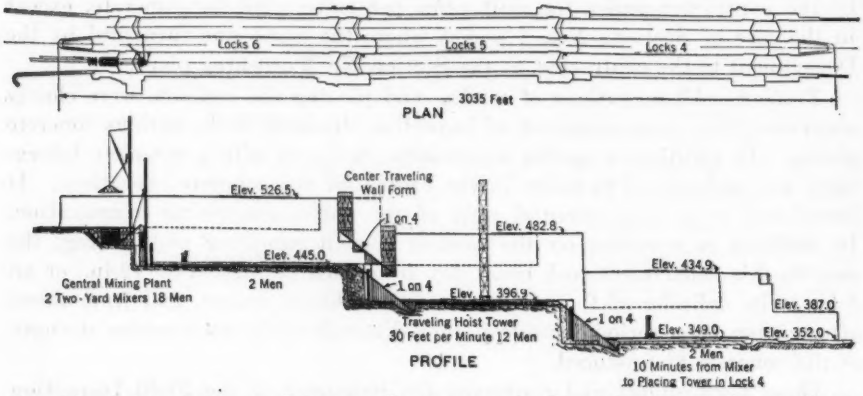

FIG. 14.—ARRANGEMENT OF CONCRETE PLANT FOR THE CONSTRUCTION OF FLIGHT LOCKS NOS. 4, 5, AND 6.

half of this wall is practically solid, and the upper half is divided into two gravity sections with a central separating space for rock-filling. The side walls of the chambers have a maximum base thickness of 47 ft.

As each of these locks has the standard lock lift of 46.5 ft., the floor level of the upper entrance of Lock No. 6 is 139.5 ft. above the floor level of Lock No. 4. Construction was started at the upper or southerly end of Twin Locks No. 6, where the central mixing plant was located.

This mixing plant consisted of duplex bins of open-frame timber construction, each with a capacity, in separate compartments, for 350 cu. yd. of sand and 430 cu. yd. of stone, with supply tracks over the tops of the bins; 15-in. radial bin gates, spaced at 6-ft. intervals, discharged the aggregate on 30-in. belt conveyors, running longitudinally under each bin, which, in turn, fed a second belt inclined at 20 degrees. The inclined conveyor elevated the materials into the stone and sand bins of the mixing plant located vertically over the mixers, these bins having capacities of 73 cu. yd. for stone and 44 cu. yd. for sand. Batch hoppers or measuring bins, each with a capacity of 80 cu. ft. and provided with separate sand and stone compartments, were fed by vertical gates in the bottoms of the aggregate bins.

Two standard non-tilting mixers, each of 56 cu. ft. capacity, and separately driven by 50-h.p. motors, at a drum speed of 12 rev. per min., were located directly under the measuring bins. They discharged directly into

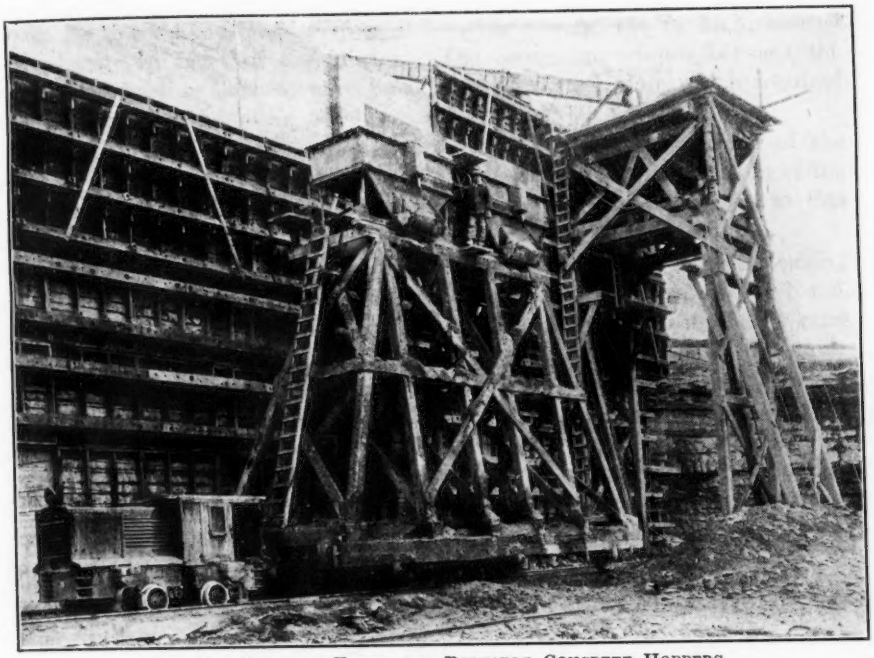

FIG. 15.—VIEW OF FIXED AND PORTABLE CONCRETE HOPPERS.

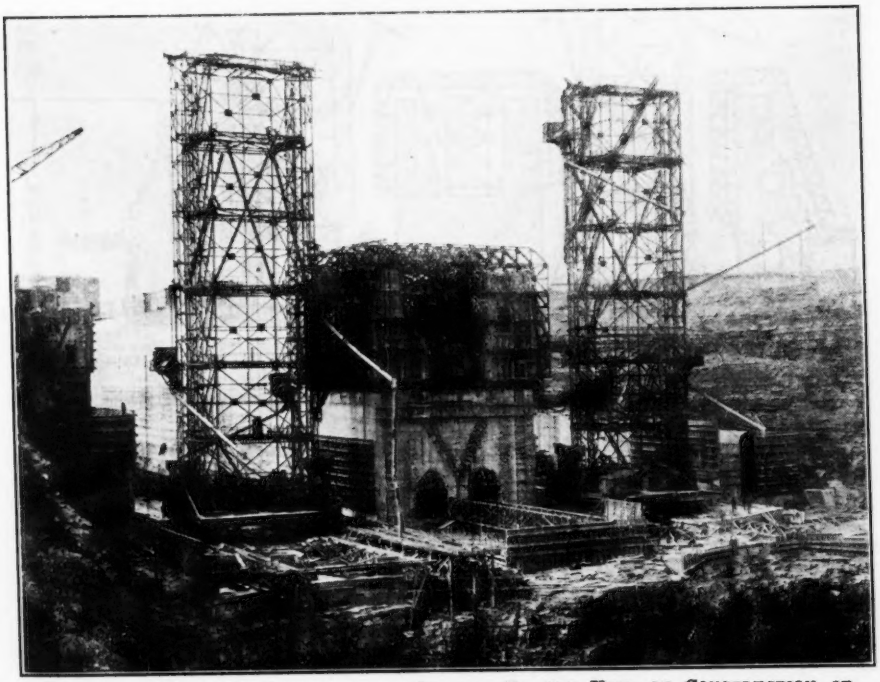

FIG. 16.—TRAVELING, HOISTING, AND PLACING TOWERS USED IN CONSTRUCTION OF FLIGHT LOCKS

60-cu. ft. tower hoist buckets in steel hoisting towers, 134 ft. high, located immediately in front of the mixers. The connecting chutes between the towers and loading hoppers were lined with $\frac{1}{8}$-in. steel plate which required replacement after handling 50 000 cu. yd. of concrete.

As the mixing plant was designed to serve for the construction of the entire flight lock structure, provision was necessary for the distribution of the concrete throughout the 3 035-ft. extent of this structure, and it is in this connection that the unusual and interesting features were introduced.

The concrete from the mixer hoist towers was passed by chutes to loading hoppers in the upper end of the east and west chambers of Twin Locks No. 6, each of 16 cu. yd. capacity, and elevated so that their bottom discharge gates were 31 ft. above floor level of the chamber.

Two trestle carriages, 23 ft. high, and fitted with three pairs of 10-ft. gauge wheels, traveling on tracks in the lock chambers, each carried two, side-dump, steel 4-yd. hoppers. Fig. 15 shows one of them being propelled by a standard-gauge gasoline locomotive, from under a fixed, 16-yd. loading hopper, at the head of the lock chambers, to the place where it discharged its concrete into the receiving hoppers of the traveling hoisting and placing towers.

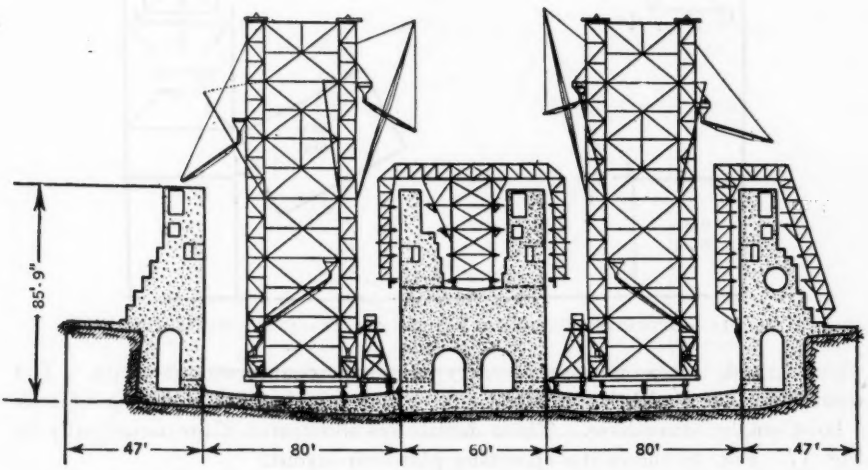

FIG. 17.—OPERATION LAYOUT FOR RECEIVING HOPPER WITH ATTACHED CHUTES.

When the concrete for Twin Locks No. 6 was completed the traveling side-dump hopper carriages were similarly utilized, successively, in the chambers of Twin Locks Nos. 5 and 4. In Locks No. 6, and, later, in Locks No. 5, their place was taken by 8-yd. traveling, bottom-dump hopper carriages propelled by 35-h.p., 3-phase, 550-volt motors geared to the rear axle and operated by one man.

The traveling, hoisting, and placing towers, one in each flight of the twin lock chambers, were of structural steel construction throughout. (See Fig. 16.)

The main tower was 30 ft. square and 150 ft. high. It was mounted on an I-beam base, 47 ft. square, with twelve cast-steel wheels, 36 in. in diameter, traveling on two lines of 100-lb. rails set at 33 ft. 9½-in. gauge. On each side of the tower, adjacent to the locks walls, a bucket hoist tower extended the full height of the main tower, and accommodated a 2-yd. hoist bucket with its lower, fixed loading hopper and upper, movable receiving hopper. To each of the movable receiving hoppers a 60-ft. boom chute was attached

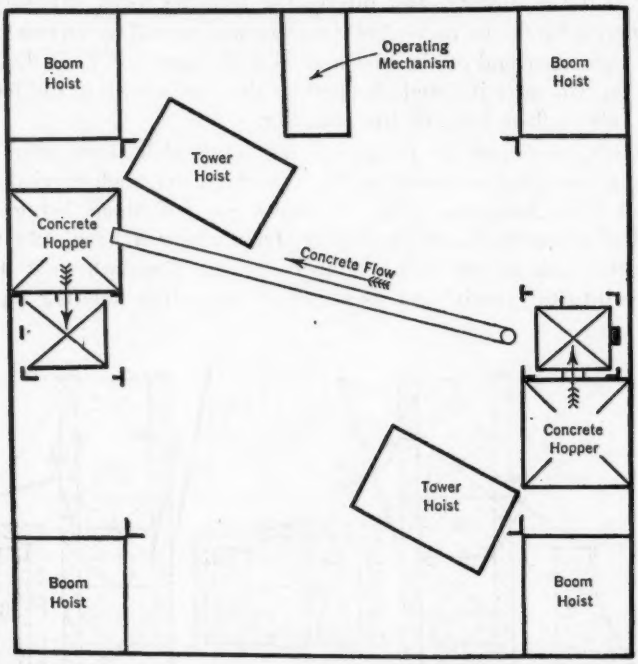

FIG. 18.—OPERATING PLATFORM LAYOUT FOR TRAVELING HOIST TOWER.

which could, when necessary, carry a 50-ft. counterweight chute. The receiving hopper with its attached chutes was raised or lowered as a unit by a hoist on the tower base. These details are illustrated diagrammatically in Fig. 17. Fig. 18 shows the operating platform layout.

A 12-yd. hopper on the side of the hoisting and placing tower adjacent to the center wall of the lock chamber, received the concrete as delivered from the mixing plant by the traveling trestle car (Fig. 15). From this hopper it was elevated by the bucket hoist on the center-wall side of the main tower to an intermediate fixed receiving hopper and then it was transferred to another 12-yd. fixed hopper on the opposite side of the main tower by inclined chutes. This second hopper supplied the bucket hoist operating on the side of the placing tower adjacent to the side wall of the lock chamber.

By means of the boom chute and counterweight chute attached to the upper movable receiving hoppers on the wall sides of the traveling placing tower the whole of the wall area at any elevation was reached, and the con-

crete was poured directly in place without any further handling. The placing tower was self-propelled at a speed of 30 ft. per min. by a chain drive from a 50-h.p. motor, and could shift from one placing position to another without delay.

On each corner of the main tower a 50-ft. timber boom operated in a vertical plane by means of a single-drum, electric hoist mounted on the tower base. These booms, with a lifting capacity of 2 tons, were used for handling chutes, forms, and other similar concrete operations.

All the equipment on each traveling placing tower was electrically operated and all machines and motors were located on the platform of the tower base (Fig. 18). This arrangement necessitated the use of 13 000 ft. of hoisting cable and the solution of a number of intricate reaving problems because of the limited space available for all the operating equipment.

The bucket hoists were fitted with a safety device mounted on the countershaft. This consisted of a series of hardened steel rollers working in tapered slots on spring mountings. Wedging action permitted the countershaft to

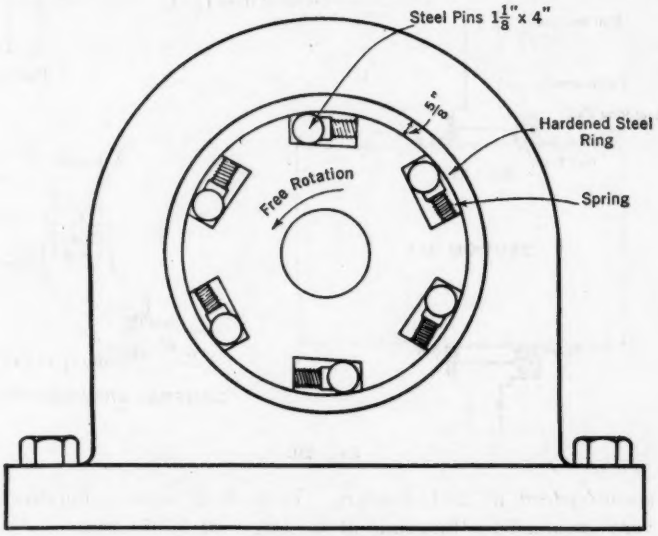

FIG. 19.—DETAIL OF SAFETY CLUTCH.

rotate in one direction only, so that in case of a power interruption the safety device automatically locked the hoist machinery and prevented the load from dropping. This safety clutch is shown in Fig. 19.

Another tower equipment detail of special interest was the cast-steel elbow by which the placing chutes were attached to the receiving hoppers of the tower hoists (see Fig. 20). It was fitted with a thimble of special design having a vertical roller and horizontal ball-bearings. The introduction of these elbows greatly increased the flexibility of the placing chutes. They functioned throughout the entire concrete operation at the flight locks without renewal.

Traveling Steel Form.—The lock walls were built in monoliths of standard 60-ft. lengths, variations from this length being made only where structural details rqeuired it.

Accordingly, the wooden forms were made up in panels, 20 ft. long by 12 ft. high, and consisted of $1\frac{3}{4}$-in. dressed fir lumber assembled on 2 by 6-in.

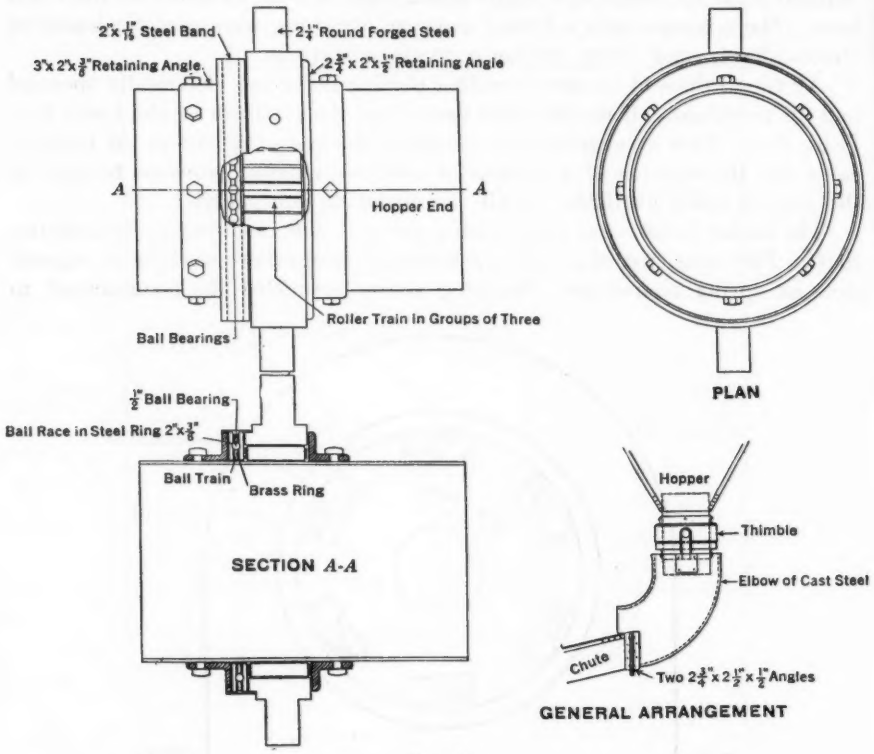

FIG. 20.

vertical studs spaced at 2-ft. centers. Four 8-in. square horizontal waling timbers were attached to the studs at an angle of 45 degrees.

Tie wires looped around steel pins set at an angle in the concrete previously placed, were brought to the outside of the form through the wales and threaded through cast-iron clamps bearing on the waling timbers. The tie wires were tightened by a special ratchet and when under the proper tension were held by a set screw in the cast-iron clamps. The wooden forms, thus secured, held a 6-ft. lift of concrete—poured continuously—in a satisfactory manner and without movement. The introduction of a 2 by 3-in. finishing strip along the top of each lift of panel form permitted the concrete being placed to a height of 2 in. above the top of the panel. By removing this finishing strip before setting the next lift of forms, a 2-in. concrete height of the finished face was exposed and provided exact alignment for the forms above. This system of forming was used in the construction of the solid

lower half of the center wall, the lower half of the west wall, and all of the east wall.

The upper half of the center wall, for 700 ft. of the lock-chamber length, consists of two separate gravity sections with a space of 24 ft. between their bases, to effect economy in concrete. It also contains the longitudinal mooring passage, the cable tunnel, and the drainage tunnel, all involving extensive form work. With a view to reducing form costs, a traveling steel form was used for this part of the work. Accordingly, a structural steel, **T**-shaped frame, 60 ft. long, was provided to carry the wooden forms outlining the two adjacent gravity walls as a unit (see Fig. 17).

This steel frame was made up of seven transverse truss sections spaced at 10-ft. centers. Each consisted of a central tower truss, 42 ft. high, an overhead section, 78 ft. long, with cantilever arms extending in a **T**-shape out beyond the wall faces. The 42-ft. vertical trusses suspended from either cantilever arm, were unified by an intermediate system of horizontal and diagonal bracing.

The wooden forms, all assembled and secured together as a unit, were suspended from this frame or gantry by turnbuckles and steamboat ratchets. The turnbuckles and ratchets, together with hardwood wedges inserted along a series of horizontal **I**-beams, provided a quick and ready means of aligning the forms for use, and they could be released quickly after the concrete of the walls was in place.

The suspended vertical trusses were tied to the central tower truss by $2\frac{1}{2}$-in., through rods, spaced 10 ft. apart in two rows, 4 ft. and 18 ft. above the base of the frame. These vertical trusses were also secured at the ends of the gantry by four horizontal steel trusses which, at the same time, carried the timber-end bulkheads forming the ends of the 60-ft. monoliths. The entire traveling form assembly was carried on a series of wheels, running on four lines of 100-lb. rail.

The longitudinal tunnels in the upper gravity wall sections, were constructed by means of collapsible steel forms, suspended from the overhead gantry by a series of 1-in. bolts and held laterally by a similar series of horizontal bolts. This traveling, 60-ft., center-wall form unit, complete with wooden face forms and enclosed tunnel forms, weighed 300 tons and was moved horizontally from monolith to monolith by a heavy winch.

Another unit of similar construction was provided for the upper gravity section of the west wall of the west lock chambers. The support, however, was two 100-lb. rails, one carried on brackets fastened to the face of the lower part of the wall (see Fig. 21), and the other laid on the anchor slab in the rear of the gravity section proper. Lateral support between the front and back vertical trusses was provided by $2\frac{1}{2}$-in through bolts and horizontal end trusses, as on the center wall form. The weight of the west wall form unit complete was 250 tons.

The traveling steel form units provided for the construction of the center and side walls did not prove to be as successful as anticipated. They were expensive to handle and set up, but the greatest expense was in transferring them from one lock to the next.

Moving Traveling Forms Between Locks.—On November 17, 1923, the traveling, center-wall form unit, weighing 300 tons, was transferred from Lock No. 6 to Lock No. 5. A timber skidway, supported at an inclination of 6 on 25, by timber bents spaced at 4-ft. centers, was constructed, extending from the unit form track level of the upper lock wall to below the similar track level of the lower lock wall. From the upper to the lower track level the vertical distance was 47 ft. 9 in. The two sets of inclined stringers were faced on their upper sides with a $\frac{1}{8}$-in. steel plate, coated with tallow, before the carrying cradle was erected.

The lowering operation was performed by two steam hoisting engines specially designed for this work and securely anchored to the center wall concrete at the lower end of the skidway. With a worm-gear reduction of 52 to 1 the engines (operating at 100 rev. per min.) gave a drum speed of just less than 2 rev. per min., and paid the cable off the drum at the rate of 12 ft. per min.

The $\frac{3}{4}$-in. plow-steel cable passed from each hoist through 18-in. single blocks secured to the concrete on the proper alignment in front of each hoist, and up the skidway below the cradle to two sets of 7-sheave, 18-in. blocks located above the form cradle.

The lower blocks were secured to the cradle with seven loops of 1-in. steel cable, wound continuously round the bridle timbers of the cradle and reaved through the head-pin of the blocks. The upper blocks were similarly held by 1-in. cables reaved through the head-pin and secured to the concrete at the top of the skidway. In order to keep the cables parallel to the inclination of the skidway, while paying out in the lowering operation, a structural steel bent fitted with a hardwood saddle was located on the concrete above the skidway with the anchor cables securing the upper sheave blocks passing over the saddle of this bent.

The becket lines from the two lowering cables were each passed through separate 3-in. pipes and heavy spiral springs, set in a concrete anchor block secured to the upper lock-wall concrete, and then around single sheave blocks, being clamped together between the sheaves. This arrangement equalized the load and synchronized the hoisting engines, while heavy cast-steel buffers clamped to the cables about 18 in. from where they passed through the 3-in. pipes and springs, provided an emergency means of automatically stopping the cradle if either lowering cable should fail.

With the lowering cable being paid off the hoist drum at the rate of 12 ft. per min., the center wall form and its supporting cradle moved down the skidway at the rate of slightly less than 1 ft. per min. The lowering operation required 4 hours.

When lowering the center wall form unit the second time, from the Lock No. 5 to the Lock No. 4 level, on August 1, 1925, a shackle on one of the cables failed. The form unit was precipitated into the east chamber of Lock No. 4 and completely wrecked. This structure was not replaced, and the entire center wall of Twin Locks No. 4 was built with wood panel forms. The two hoisting and placing towers were successfully lowered to the floor

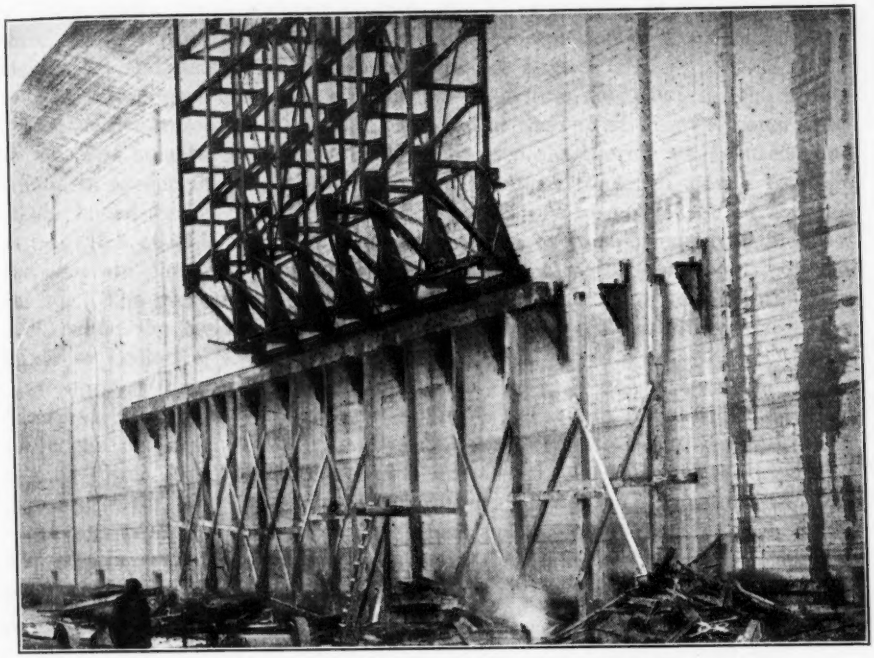

FIG. 21.—BRACKET SUPPORT FOR TRAVELING FORM UNIT.

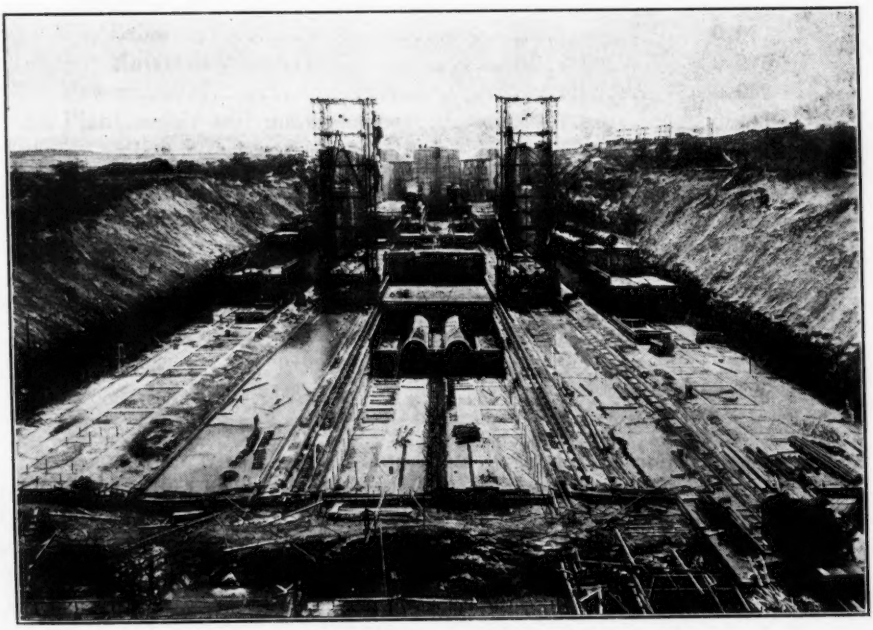

FIG. 22.—CONSTRUCTION VIEW IN LOCK NO. 4.

of Twin Locks No. 4 without mishap, but after the loss of the center-wall form, the west wall traveling form, with the completion of Lock No. 5, was dismantled and its further use abandoned (see Fig. 22).

Concrete Output in the Flight Locks.—Concrete operations in the flight-lock structure were begun in July, 1922, and at the end of the 1927 concreting season, this structure was practically completed. Concrete was poured continuously except for the winter months, December to April. In a total of 13 511 hours worked, 999 700 cu. yd. of concrete were poured at an average rate of 73.3 cu. yd. per hour. In other words, over a period of 46 months, the concrete plant maintained an average output, per 10-hour shift, of 733 cu. yd. The maximum average output per shift of 849 yd. was obtained in 1926, when the concrete was being moved the greatest distance—from the mixer to Lock No. 4.

Concrete Costs.—The approximate total cost of the concrete plant as described herein was $475 000. The mixing plant and its equipment at the south end of Locks No. 6 cost about $70 000; each traveling, hoisting, and placing tower, $75 000; the center wall form unit, $90 000; and the west wall traveling form, $50 000; while the operation of lowering a traveling tower or form unit from one lock level to the next below, cost $7 850 for each move. The cost per cubic yard of the concrete in the flight-lock structure, exclusive of overhead and plant rental, was as follows:

Mixing and Placing:	
Labor	$0.490
Materials	1.375
Forms:	
Labor	0.577
Materials	0.275
Power	0.062
Plant repair and maintenance	0.141
Total	$2.920

The operation of the concrete plant was carried on by eighteen men at the mixing end, six men operating the transfer hopper carriages through the locks, and twelve men on the two hoisting and placing towers. (See Fig. 14.) This is exclusive of carpenters, riggers, and helpers employed on forms, etc.

The unit costs of forms per cubic yard of concrete were:

All types	$0.82
Wooden forms	0.69
Steel forms	1.15

In connection with the foregoing costs it should be noted that, on that part of the flight-lock structure where steel forms were used, the area of form work per cubic yard of concrete was slightly more than twice that of the structure where wooden forms were used. Furthermore, the center wall form unit was used only for the placing of 220 000 cu. yd. of concrete in Locks Nos. 6 and 5, while had it not been destroyed by the accident, it would have accommodated an additional 110 000 cu. yd. of the center wall of Lock No. 4.

For the concrete placed in the flight lock structure the Department paid the contractor at his unit price of $4.14 per cu. yd. and supplied, as well, the cement which cost the Department between 1922 and 1928 an average price of $2.21 per bbl. There were 1⅓ bbl. of cement used per yard of concrete in this structure.

Concrete Cribs

The use of reinforced concrete cribs instead of the usual type of timber cribs for under-water foundations in the Harbors of Port Weller and Port Colborne and in the summit level of the Welland Ship Canal presents some interesting construction features.

Cribs for Harbor Breakwater.—The artificial harbor at Port Weller, built by projecting two embankments out into Lake Ontario, required retaining walls to establish definitely, the outer entrance to the harbor as well as the inner docking space. These walls were founded 35 ft. below lake level on a clay bottom.

Reinforced concrete cribs, 109 ft. 8 in. long by 37 ft. 8 in. wide, and 34 ft. deep, of cellular construction, were provided for the portion of the retaining wall section up to lake level. Their construction afloat was effected in the following manner: A timber pontoon, 111 ft. 6½ in. long by 39 ft. 2½ in. wide, and 12 in. deep, of rigid mill-floor frame construction, was built. It was fitted with removable timber sides, 7 ft. high, which were secured to the pontoon bottom by means of vertical through bolts. The bolts were spaced at 6-ft. centers around the perimeter of the pontoon and were fastened into special castings set in the bottom. The tops projected sufficiently above the pontoon sides to permit their square ends being turned up tight with wrenches. To insure a water-tight joint between these removable sides and the pontoon bottom a tongue on the bottom edge of the side sections was fitted into a groove in the upper side of the pontoon bottom. At the corners the miter junction of the pontoon sides was secured by bolts held by special lug fittings attached to the sides. When the pontoon was assembled and with all the joints caulked, it represented a large water-tight box capable of carrying an interior load which could sink it in the water to a depth of 7 ft., while it still remained afloat.

The interior dimensions of the pontoon with the sides erected, corresponded to the over-all length and width of the concrete crib built therein and so for a height of 7 ft. it served as the external form work. For the remainder of the crib, panel forms of convenient size were utilized in the construction of the exterior and interior cross-walls.

By locating the concrete mixing plant immediately adjacent to the face of the dock where the crib was being constructed, and by equipping it with a tower hoist and a chute flexible enough to reach the full width of the crib, all points in the length of the crib were reached by the simple expedient of floating the whole structure past the concrete plant. Furthermore, by the use of two pontoons, the continuous construction of several concrete cribs could be carried on at one time.

By the time the weight of concrete submerged the carrying pontoon to within 6 in. of the top of the 7-ft. side walls, the concrete was about 12 ft.

above the floor level, and the concrete crib itself with this height of wall was capable of remaining afloat without the assistance of the pontoon.

A rectangular opening was left in the bottom of each cell to permit filling when the crib was sunk to its final position. Each of these rectangular openings was closed by removable timber bottoms, while the crib was still under construction and supported by the pontoon. The bottoms were securely wedged down and held in place by knee-braces so as to be water-tight during the construction of the concrete crib and until finally it was sunk in its permanent position.

When the concrete was of sufficient height to permit the crib to remain afloat unassisted, the removable sides of the pontoon were released. By means of concrete blocks placed on the bottom of the pontoon below the timber bottoms, before these latter were put in position, the pontoon was sunk clear of the bottom of the concrete crib and taken away for use in starting the construction of another crib. From this point on, the construction of each crib was merely a matter of erecting forms and steelwork and pouring concrete until the full height of the crib was obtained.

The initial operation of constructing the cribs for Port Weller Harbor was done in the Harbor of Port Dalhousie, about 4.5 miles west of the point where the cribs were used. This was necessary because no protected area was available at first at the new harbor site. Accordingly, these first cribs, when fully completed, were towed by tug from Port Dalhousie to their site at Port Weller where they were sunk immediately on a previously prepared foundation.

Preparing Foundation for Harbor Cribs.—The foundation for the cribs was prepared by first excavating over the area to be occupied to a depth of 2 ft. below the bottom level of the crib. On this excavated foundation area

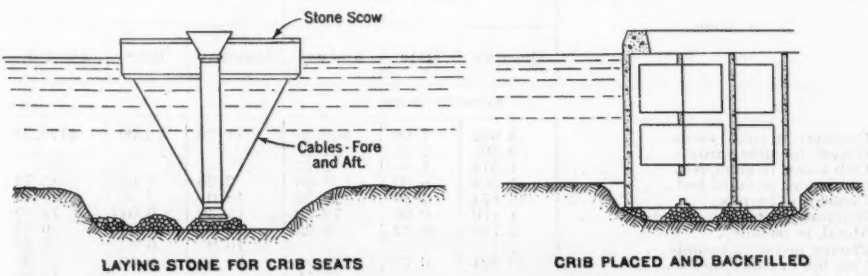

FIG. 23.—SKETCHES SHOWING METHODS OF PREPARING BOTTOM FOR BREAKWATER CRIBS.

windrows of crushed stone were placed by means of a pipe with a flared lower end. This was suspended from a scow with the lower end of the pipe carried at the elevation of the crib bottom. The windrows were placed to correspond with the longitudinal side and center walls of the crib itself, and by moving the scow along, during the process of placing the stone through the pipe, they were all leveled off at the required elevation (see Fig. 23). The completed crib was floated to a position over its final location and was securely held by a dredge while being sunk by filling with water.

As soon as a crib was landed in its final position the wedges securing the removable timber bottoms were removed by trip chains, allowing the timber bottoms themselves to float to the surface. The process of filling the crib with clay or rock was then begun. With the cribs entirely filled and after the filling was satisfactorily settled, the top of the crib was sealed with concrete and the monolithic superstructure wall of gravity section erected on it.

Fourteen concrete cribs were used in outlining the outer entrance at Port Weller Harbor, while thirty additional cribs of this type provided the substructure of part of the inner dock walls. At Port Colborne a breakwater, 2 000 ft. in length, was provided for the protection of the entrance to the harbor, using reinforced concrete cribs built and placed with a concrete superstructure, in a manner similar to that previously described. However, as at Port Colborne, the breakwater cribs were founded entirely on the rock and only sufficient of this rock was removed to provide a level surface on which to place each crib, the heights of the cribs varied according to their location in the breakwater, and cribs, without removable timber bottoms, were used.

Cribs for Breakwater at Port Colborne.—The harbor space at Port Colborne was limited and the maximum depth of water there was not more than 19 ft. Therefore, the construction of the cribs afloat, using the same pontoon procedure as previously described, was followed in the Harbor of Port Maitland, twenty miles west of Port Colborne, on Lake Erie. After completion,

TABLE 10.—Comparative Cost per Linear Foot of Reinforced Concrete Crib Docks

Item	Port Weller			Port Colborne		
	Quantity	Rate	Amount	Quantity	Rate	Amount
Substructure						
Concrete, in cubic yards	8.642	7.00	$60.49	11.75	15.00	$176.25
Gravel, in cubic yards	9.505	2.60	24.71	.....	.....	.....
Crib stone, in cubic feet	1.513	1.50	2.27	.....	.....	.....
Crib stone, in cubic feet	1.513	6.00	9.08	2.25	7.00	15.75
Cement, in barrels	13.874	2.20	30.52	18.0	2.30	41.40
Reinforcing steel, in pounds	1.170	0.06	70.20	15.30	0.04¾	72.60
Metal, in pounds	2.742	0.12	0.33	10.9	0.05	0.55
Placing metal, in pounds	.....	.....	.....	10.9	0.02	0.22
Crib fill, in cubic yards	40.324	0.75	30.24	35.3	0.50	17.65
Cost of towing, placing	.....	.....	13.51	.....	.....	11.30
Cost per linear foot	.....	.....	$241.35	.....	.....	$335.72
Superstructure						
Mass concrete, in cubic yards	1.585	5.25	$8.32	4.7	4.70	$22.10
Reinforced concrete, in cubic yards	0.870	7.00	6.09	.....	.....	.....
Crib stone, in cubic feet	2.000	1.50	3.00	.....	.....	.....
Cement, in barrels	3.683	2.20	8.10	9.	2.30	20.70
Rock fill, in cubic yards	7.500	0.75	5.62	8.6	0.50	4.30
Metal, in pounds	0.060	0.35	0.02	.....	.....	.....
Reinforcing steel, in pounds	30.0	0.06	3.60	.....	.....	.....
Cost per linear foot	.....	.....	$34.75	.....	.....	$47.10
Total cost of dock per linear foot	.....	.....	$276.10	.....	.....	$382.82

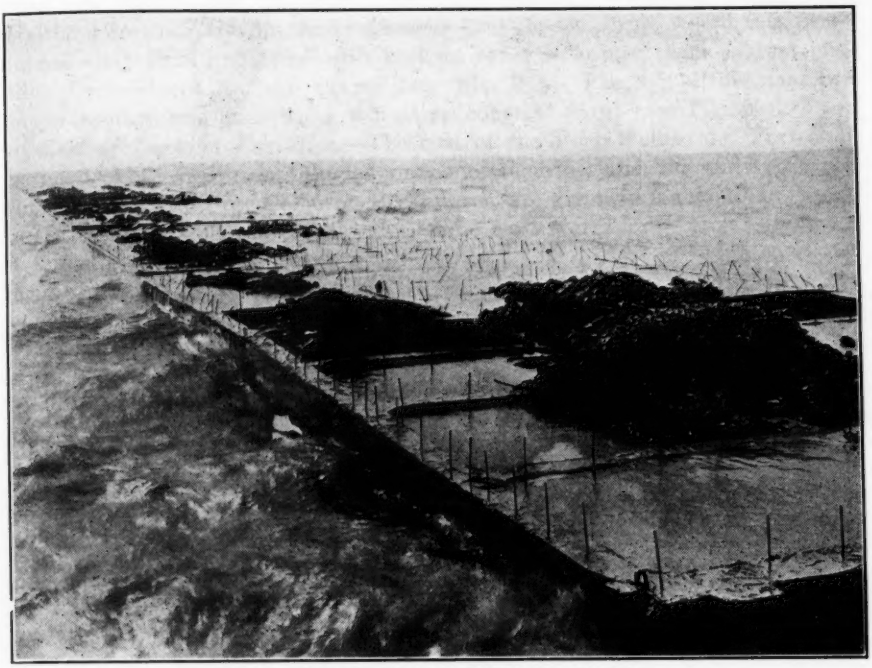

FIG. 24.—REINFORCED CONCRETE CRIBS IN PLACE.

FIG. 25.—BREAKWATER AT PORT COLBORNE READY FOR SUPERSTRUCTURE.

the cribs were towed 20 miles from their point of construction at Port Maitland to their site in the breakwater at Port Colborne, where they were immediately sunk and filled with rock in order to secure them against possible displacement by lake storms (see Fig. 24). The monolithic concrete superstructure was built using a floating concrete plant (see Fig. 25).

Cost of Docking Facilities.—The cost of the Port Weller and Port Colborne docking facilities, founded on concrete cribs, and the unit cost per linear foot of the Port Colborne Breakwater, are given in Table 10.

In the Port Colborne Breakwater, these costs were:

Substructure per linear foot.....	$332.79
Superstructure per linear foot.....	109.70
Total	$442.49

The breakwater cost is higher than that of the docking facilities because of the larger cribs, the heavy concrete superstructure, and the heavy stone fill on the seaward side.

Cribs for Construction of Bridge Piers.—An even more interesting use was made of the reinforced concrete crib in the construction of the bridge substructures on the summit level of the canal between Thorold and Port Colborne.

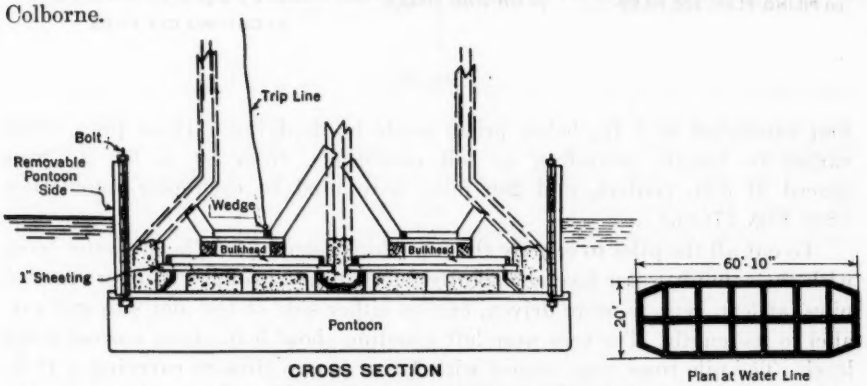

FIG. 26.—CRIB CONSTRUCTION ON PONTOON.

The summit level of the Third Canal was widened and deepened to the enlarged section of the Ship Canal, and, in consequence, all brige piers were built in the water, in order to avoid very difficult and expensive unwatering operations. Vertical lift bridges of 200 ft. clear span were provided for all railway and highway crossings of the summit level of the Ship Canal, and the main piers of these bridges are of rectangular section approximately 65 ft. long and 30 ft. wide, with an average height of 38 ft.

In order to eliminate the necessity of using deep coffer-dams an alternative method was adopted, by which cellular reinforced concrete caissons, with exterior dimensions the same as the bridge piers, were built, afloat. In building these cribs, a timber pontoon with removable sides of the exact base dimensions of the pier, was used, following closely the procedure for the construction of the harbor cribs. (See Fig. 26.) In this case, as at Port Weller,

removable timber covers were provided in the bottom opening of each cell of the crib. After the crib was placed and the covers were removed, the concrete filling would occupy the space below and around the supporting foundation piles, bonding the pier to its foundation (see Fig. 28).

As rock at the bridge sites on the summit level was at considerable depth below Ship Canal grade, the piers were founded on timber bearing-piles driven to refusal. For this purpose a floating pile-driver, equipped with telescopic leads and a submersible steam hammer, was used. The pier area was

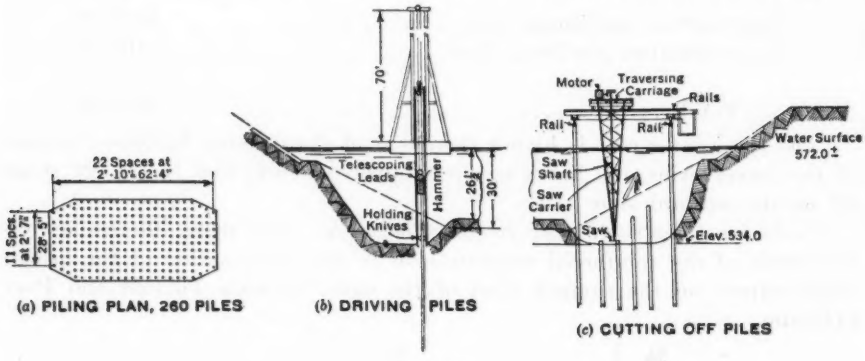

FIG. 27.

first excavated to 5 ft., below prism grade by dredging. These piles which varied in length, according to soil conditions, from 30 to 60 ft., were spaced at 3-ft. centers, and 260 piles were used in each pier foundation. (See Fig. 27(*a*).)

To cut all the piles to exactly the same level, about 35 ft. below water level, with the summit water level constantly fluctuating with Lake Erie, two rows of piles, at 5-ft. centers, were driven, one on either side of the pier site and parallel to its length. The tops were left standing about 5 ft. above normal water level. The pile rows were capped with 12-in. square timbers carrying a 40-lb. rail. A timber carriage with double-flanged wheels ran on the rails of the pile bents. This carriage, in turn, was similarly fitted on top with a pair of rails on stringers located transversely to the length of the pier. Traveling on the upper track was a second carriage to which was attached the upper end of a structural steel frame of triangular section, 48 ft. long and hanging down into the water. Attached to the one vertical face of this frame, by a series of bearings, was a 3½-in. steel shaft suspended by a thrust-bearing carried by the deck of the upper carriage. To the lower end of this shaft a 52-in. circular saw was mounted on a horizontal plane, and on the upper end, above the thrust-bearing, was attached a 23-in. crown pulley belt driven by a 75-h. p. electric motor on the upper carriage deck. The arrangement is shown diagrammatically in Fig. 27(*c*).

In operation, the submerged saw was started clear of the pile foundation area. The foundation piles were cut off at the required level by moving the

lower and upper carriages lengthwise and crosswise as required. The average time taken to cut off all the piles in a pier foundation by this method was 40 hours.

A pier caisson was constructed on its pontoon until, with the walls built up about 12 ft., it had sufficient buoyancy to float by itself. The removable sides were then released, and the pontoon bottom dropped away from the crib with the weight of concrete blocks placed on the pontoon under the removable bottoms, before the caisson construction was started. Three of these operations are shown in Fig. 28.

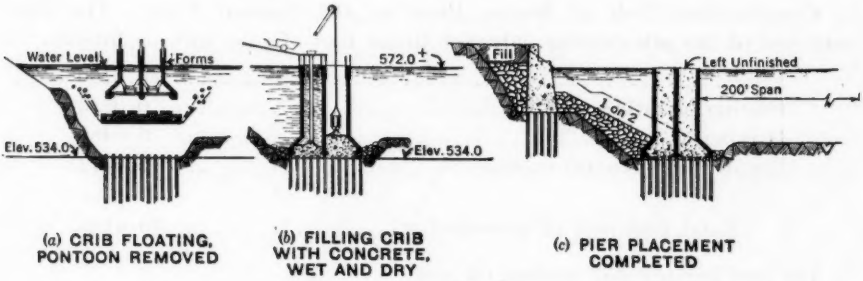

FIG. 28.

During the period of 2 to 3 hours that the caisson setting was under way, canal navigation was not allowed to pass the site. This permitted the use of setting tackle, anchored to either side of the prism, and also prevented the surge created by a passing vessel from disturbing the alignment of the caisson when landing on its foundation. All caissons built and placed by this procedure were landed within 3 in. laterally and longitudinally of their theoretical positions.

Immediately after a pier caisson was placed on its foundation piling and completely filled with water, coarse stone, from the excavation, was dumped by scow around the outside, to secure it further and to prevent the concrete filling from escaping. The removable timber bottoms were next released by removing the wedges and knee-braces with a trip line operated from above water level. (See Fig. 26.)

With the bottom bulkheads and sheeting removed and the caisson base outlined with stone filling, all cells of the caisson, as well as the space beneath and around the supporting piles, were filled to a height of 10 ft. above the caisson bottom, with concrete placed under water by means of a tremie or bottom-dump bucket (see Fig. 28(*b*)). This weight of concrete in the bottom of the caisson permitted the cells to be unwatered, one at a time, and the remainder of the concrete filling placed in the dry.

To allow for any possible settlement after the concrete had been poured, the top of the pier was left unfinished until immediately preceding the erection of the bridge superstructure. This precaution proved unnecessary, as no settlement was observed at any of the piers after the bottom concrete filling was in place. The weight superimposed on each of these bridge piers, by the steelwork of a vertical lift bridge, is 1 100 tons.

Six working days were required to assemble the pontoon with its removable sides and place the bottom forms and reinforcement steel ready for the first pour of concrete. Seventeen working days were required to pour the concrete of the caisson to its full height and to sink the completed caisson on its foundation.

Seventeen more working days were required after the caisson was placed on its foundation to remove the bottom bulkheads and sheeting, to seal the cell bottoms with under-water concrete, and then to unwater the caisson and pour the remainder of the concrete "in the dry."

Construction Costs of Bridge Piers on the Summit Level.—The total field cost of the pile-driving only, per linear foot of pile, was, as follows:

Piles	$0.3100
Peeling, pointing, rafting	0.0233
Driving	0.0480
Repairs and general expense	0.0372
Total field cost of pile-driving	$0.4185

The cost per pile for cutting off was:

Erection of cut-off outfit at pier site	$1.064
Operating cut-off outfit	2.159
Dismantling and removing forms	0.396
Pile cut-off outfit, first cost	2.353
Total cost of cutting off one pile	$5.972

The foundation piling varied in length from 30 to 60 ft., but the cost of "cutting off" bears no relation to the length of piles used. The entire value of the cut-off outfit is absorbed in the cost of cut-off given previously because it cannot be considered plant in the ordinary sense, its further use elsewhere being highly improbable.

The cost per cubic yard of concrete in the piers was, as follows:

Assembling and unwatering pontoon	$0.95
Forms	1.09
Gravel	1.98
Handling cement (supplied by Department)	0.09
Handling reinforcement steel	0.49
Mixing and placing	2.22
Total field cost	$6.82

One pontoon and one set of 1-in. panel forms, costing $11 000, served for the construction of ten piers.

The contractor was paid at the following unit prices by the Department for the various items of work entering into the construction of the bridge piers:

Mass concrete per cubic yard	$8.00
Timber bearing-piles per linear foot	1.25
Rock-fill around pier per cubic yard	1.25
Steel reinforcement per pound	0.05
Excavation (earth) per cubic yard	0.42

In addition, the Department paid the contractor a bulk sum of $40 000 for each bridge substructure, as completed, to cover the additional cost of excavation and all other operations involved in the substructure construction.

Chippawa Creek Siphon Culvert

A siphon culvert was provided on Section No. 6 at Welland, for the passage of Chippawa Creek under the Ship Canal prism. This presented one of the most interesting general problems solved in the construction of the Welland Ship Canal. Fig. 29 is the plan view of this crossing, showing

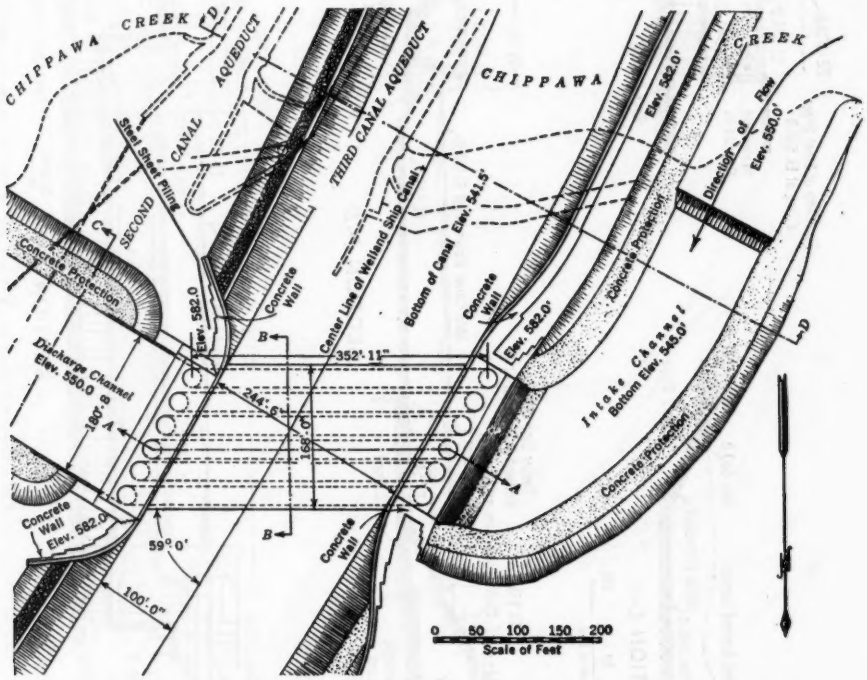

Fig. 29.—Plan of Siphon Culvert in Section 6.

over-all dimensions and general characteristics. Details essential to an understanding of the problem are shown in Fig. 30.

The structure extended for nearly one-half its length out into the prism of the Third Welland Canal. This end had to be enclosed by a cellular steel sheet-pile coffer-dam (see Fig. 31).

Steel Sheet-Piling.—Borings taken over the site before the final design was adopted, showed the rock surface at an average elevation of 483.5,

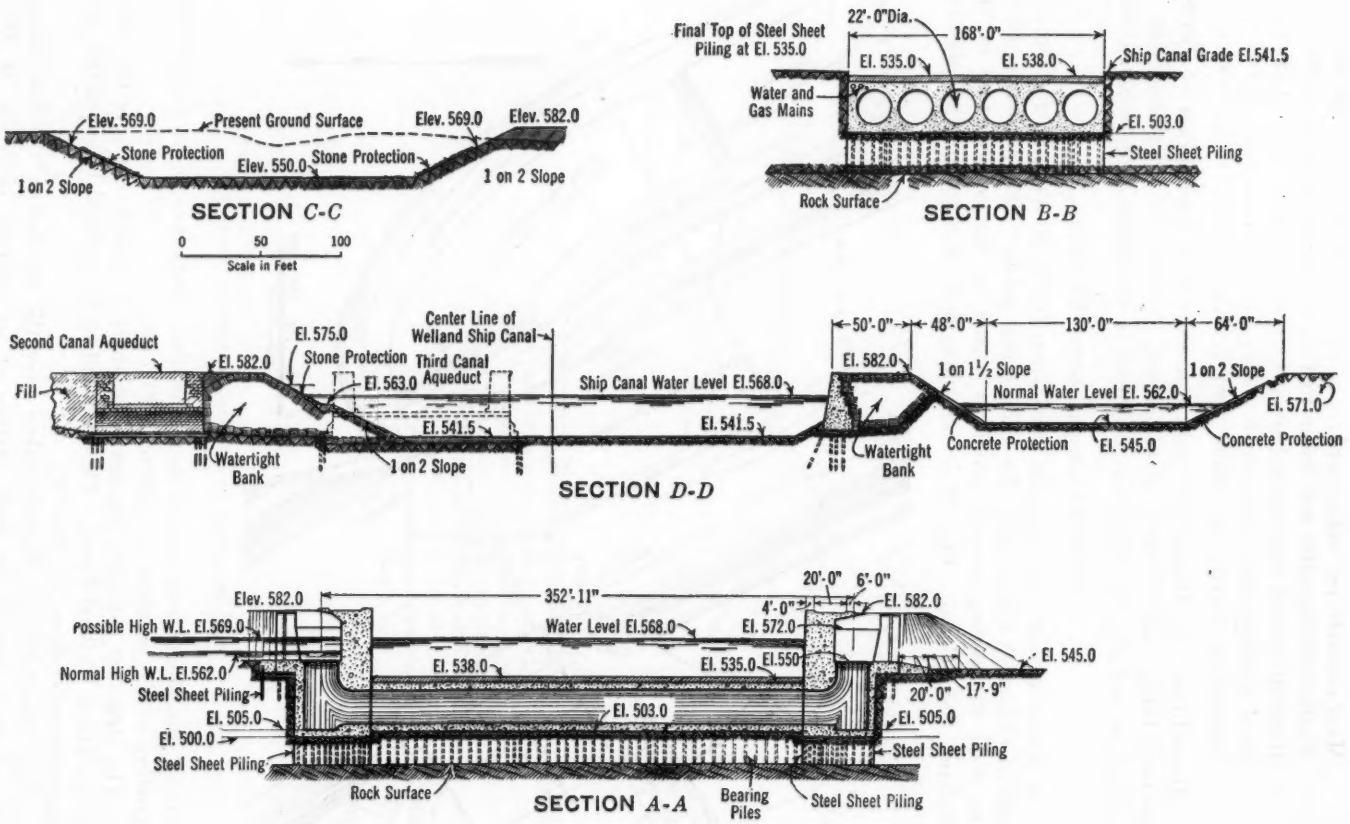

FIG. 30.—SECTIONAL VIEWS OF THE SIPHON CULVERT.

approximately 100 ft. below the ground surface, or 88 ft. below the normal water level of the Third Welland Canal. The overlying material was chiefly a soft clay, with 5 or 6 ft. of sand and gravel immediately overlying the rock.

The coffer-dam consisted of twenty-six cells with interior and exterior faces arched to a radius of 36 ft., and the adjacent sides straight. The con-

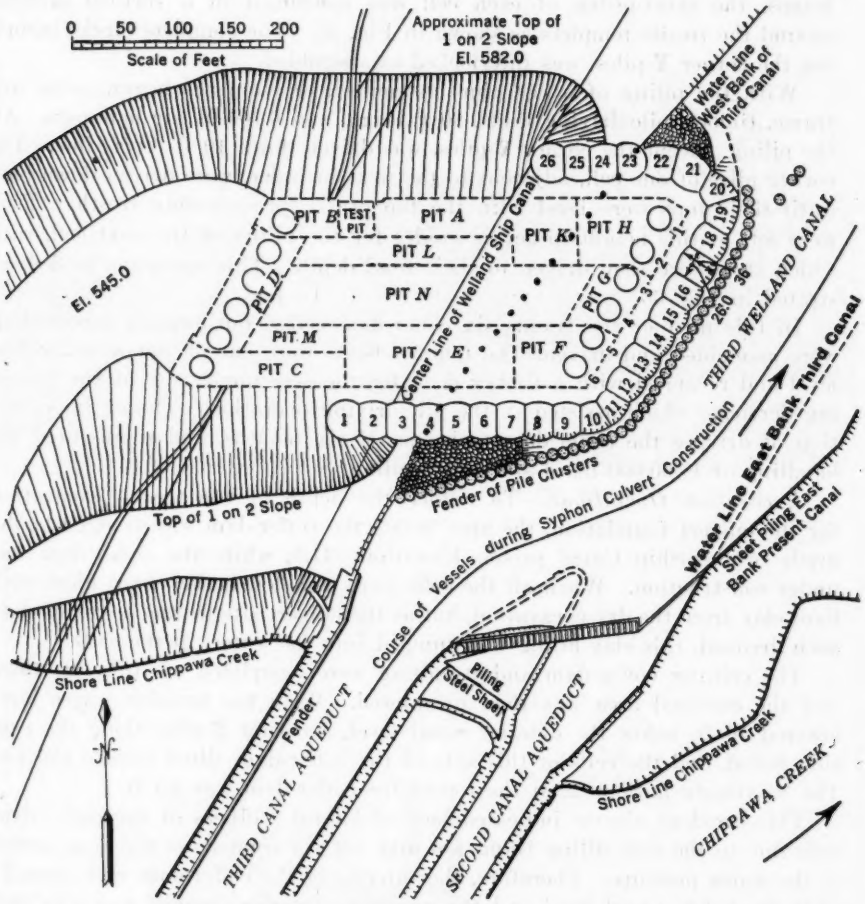

FIG. 31.—ARRANGEMENT OF EXCAVATION PITS AND COFFER-DAM CELLS, CHIPPAWA CREEK SIPHON.

nection between the arched sides and the straight sides was made by the use of fabricated Y-piles, thus forming a continuous connection between all cells.

Straight web interlocking steel sheet-piling of 15-in. by $\frac{3}{8}$-in. and $12\frac{3}{4}$-in. by $\frac{3}{8}$-in. sections was used. The piling in the exterior wall was 70 ft. long, and that in the inner and partition walls, 55 ft. long. It was driven by the pre-assembly method.

Pre-Assembly Method of Driving Steel Sheet-Piling.—First, three wooden pile-bents were driven in the interior area of each cell normal to the length of the coffer-dam, with their tops 30 ft. above the final level to which the sheet-piling was driven. Templets of the cell outline were attached to these pile-bents, one at the top and one at the ground or water-line.

Using traveling steel stiff-leg derricks of 10-ton capacity, fitted with 80-ft. booms, the sheet-piling of each cell was assembled in a vertical position around the trestle templets as shown in Fig. 32. The complete circle, including the corner **Y**-piles, was interlocked as assembled.

With the piling of a cell thus assembled, driving was begun, with stiff frame, timber, pile-driving leads, 80 ft. high, and 6 700-lb. air hammers. All the piling, except the corner **Y**-piles, was driven down 10 ft. at a time. The corner piles of one cell, adjacent to the next in order, were driven down only until their tops were level with the top of the pre-assembly trestle. They were left at this height to act as guides for the piling of the next cell, with which they were then driven to their final depth. This operation is demonstrated in Fig. 33.

In this manner the twenty-six coffer-dam cells, continuously interlocked, were assembled and driven. The templet bents were cut off below coffer-dam level and recapped with a timber deck for the accommodation of the traveling derricks, which assisted in the pile-driving operation. The entire operation of driving the coffer-dam took exactly six months, and represented the handling of 1 700 000 lb. of steel sheet-piling.

Excavation Operations.—To reduce the depth of the trench excavation for the culvert foundation, the area inside the coffer-dam was dredged to the grade of the Ship Canal prism, Elevation 541.5, while the coffer-dam was under construction. When all the cells were completed, they were filled with hard clay from the dry excavation, but as the culvert site in the meantime had been dredged, this clay filling was dumped into the water, in the cells.

The cellular coffer-dam and its filling were completed on July 31, 1926, and the enclosed area was then unwatered. With the interior water level lowered 20 ft. below the exterior canal level, a corner **Y**-pile, along the east side failed, and the cells of this side of the coffer-dam tilted toward the pit. The maximum movement at the top of the coffer-dam was 4.5 ft.

This accident clearly indicated lack of lateral stiffness of the coffer-dam cells due to the clay filling being wet and not yet compacted when subjected to the water pressure. Therefore, the interior of the coffer-dam was immediately flooded to canal level and the necessary interior support was provided by depositing gravel against the inner side.

The interior level was then lowered (beginning October 6, 1926) at the rate of 3 ft. in 24 hours. This slow rate allowed the coffer-dam to be observed for further deflection, and it also allowed the water to drain from the supporting gravel. No further deflection occurred.

Test borings indicated the material to be penetrated below Ship Canal grade, in the interior of the coffer-dam, to be soft clay. The foundation construction was therefore carried on within steel sheet-pile enclosures, securely braced with timber as excavated.

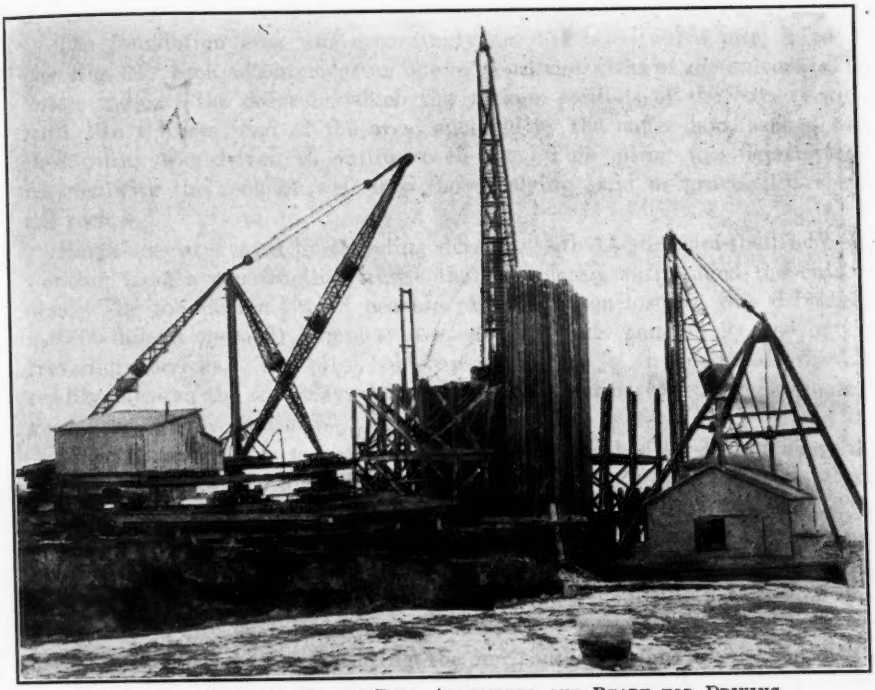

FIG. 32.—CELLULAR COFFER-DAM, ASSEMBLED AND READY FOR DRIVING.

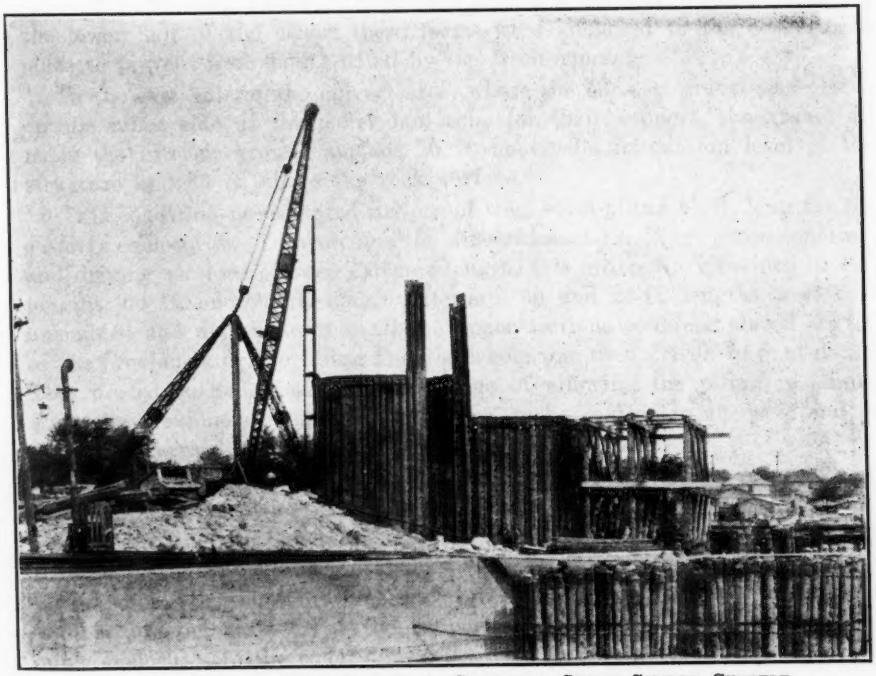

FIG. 33.—DRIVING SHEET-PILING AT CHIPPAWA CREEK SIPHON CULVERT.

The foundation area was accordingly divided into twelve pits, *A* to *N* (see Fig. 31), each accommodating one or more monoliths of the culvert. The letters indicate the order in which the various sections of the culvert were built. In the west end of the area, enclosed by the coffer-dam, arched web sheet-piling was driven to outline each pit. This piling was driven to a contact with the rock or well into the overlying sand or gravel bed above the rock.

Earth was excavated by traveling derricks with 1½-yd. clam-shell buckets, working from a construction trestle that completely surrounded the culvert area. The foundation timber bearing piles in all enclosures were driven by a 4 000-lb., air-operated hammer in suspended leads handled by one of the traveling derricks. The piles, which averaged 20 ft. in length, drove very readily through the soft clay, down to the gravel overlying the rock where a good bearing was obtained.

The piles were then cut off by hand at the prescribed level, 1 ft. above the bottom of the excavation. Because the pits were enclosed by sheet-piling, the piles driven into the foundation raised the bottom an average of 2 ft. above the level to which it was taken out originally. A second clean-up was necessary, therefore, before placing the concrete; but this second operation was later eliminated by excavating each pit to a depth of 2 ft. below final grade before any bearing piles were driven.

In the enclosure incorporating the horizontal sections of the tubes, the forms of 2-in. lagging on segmental ribs, were made up in sections corresponding to the vertical distance between frames of bracing. When placed for the lower half of the circle, these forms were anchored to the concrete in place to prevent their being lifted by the fresh concrete.

In the east half of the culvert area, where the bank of gravel was placed on the inner side of the coffer-dam cells for their support, the gravel fill made the interior ground surface 70 ft. above the foundation level of the structure and 83 ft. above the rock surface.

This condition necessitated the use of steel sheet-piling 83 ft. long for the easterly enclosures. To overcome the difficulties of handling, pre-assembling, and driving sections of such extreme length, this piling was provided in two lengths, 60 ft. and 23 ft. long. Alternate 60 and 23-ft. lengths were first assembled and driven down until the upper sections could be placed on top of the previously driven piling and the whole was then driven to final depth. This procedure had the added advantage of allowing the piling reclaimed from the previously completed enclosures, to be re-used in the east end of the culvert area.

It was important to maintain the supporting power of the gravel bank, as the pits adjacent to it, along the east and south sides of the culvert area, were being excavated, and the concrete was being placed. As removed, the gravel from these outer enclosures was piled in similar form, as originally placed, along the sides of these enclosures away from the outer coffer-dam. Timber bracing transferred the support of this gravel bank through the outer enclosures to the coffer-dam cells.

When the enclosure of Pit *H* in the northeast corner of the culvert area was driven and the interior excavation removed to within a few feet of final grade, the piling of the west wall settled vertically a maximum of 4 ft. at its mid-distance (see Fig. 34). This settlement was due to two conditions: (1) The piling of the wall was not driven to contact with the rock; and (2) in driving, this west-wall piling sloped outward at its bottom, so that with the interior excavation removed the sloped wall carried the vertical load of the exterior material. The waling timbers and their supporting diagonal struts along the west wall settled with the wall piling. This concentrated the exterior loads on the undisturbed central tiers of square struts, loading some of them in the bottom three frames to destruction. By immediately introducing vertical supports under the bracing of the west wall and replacing the failed struts with new timbers, settlement was stopped, but not before the lower waling timbers along the north side of the enclosure were bowed in as much as 4 ft. in a length of 12 ft. by the movement of the north wall. These waling timbers remained intact, however, until reinforced with new struts. Had the outline of Pit *H* been such as to allow the entire use of struts normal to the faces supported, it is most probable that no timber failures would have occurred. By driving the steel piling to rock no settlement would have taken place.

With the concrete in the enclosures around the outside of the culvert area completed, the construction of the horizontal portion of the two interior tubes, 3 and 4, in Pit *N* (see Fig. 31), was the last operation.

Pit *N* accommodating these tubes was outlined on its north and south sides with the sheet-piling of the previously driven enclosures. The heavy pressures experienced in the outer enclosures demonstrated that continuous support by timbering was necessary across the pit from its north to its south side. Therefore, inclined timber bracing had to be carried through the enclosure, *N*, from the higher level of bracing in Pit *E*, along the south side of the coffer-dam, to the similar bracing, at a lower level, above the finished concrete in Pits *A*, *B*, *K*, and *L*, on the north side (see Fig. 35).

A very convincing demonstration of the severity of the external pressure exerted, was given when the completed concrete monolith in Pit *F* in the southeast corner of the culvert area weighing about 12 000 tons was moved as a unit approximately 2 ft. horizontally from its original position by the pressure of the adjacent portion of the outer coffer-dam. This monolith comprised one-third of the massive east head-wall.

This occurrence fully determined the necessity of introducing the through-bracing across the entire culvert area without disturbing the bracing already in place. To accomplish this through the sheet-piling in place on the north and south sides of Pit *N*, the piling was burned off opposite each inclined strut down to the bottom of the first frame level, and through these gaps the intermediate bracing was introduced, as shown in Fig. 36. This procedure was continued to the bottom of Pit *N*, as rapidly as the excavation permitted.

After all concrete in the culvert structure proper had been completed, the interior through-bracing was no longer required, because the prism

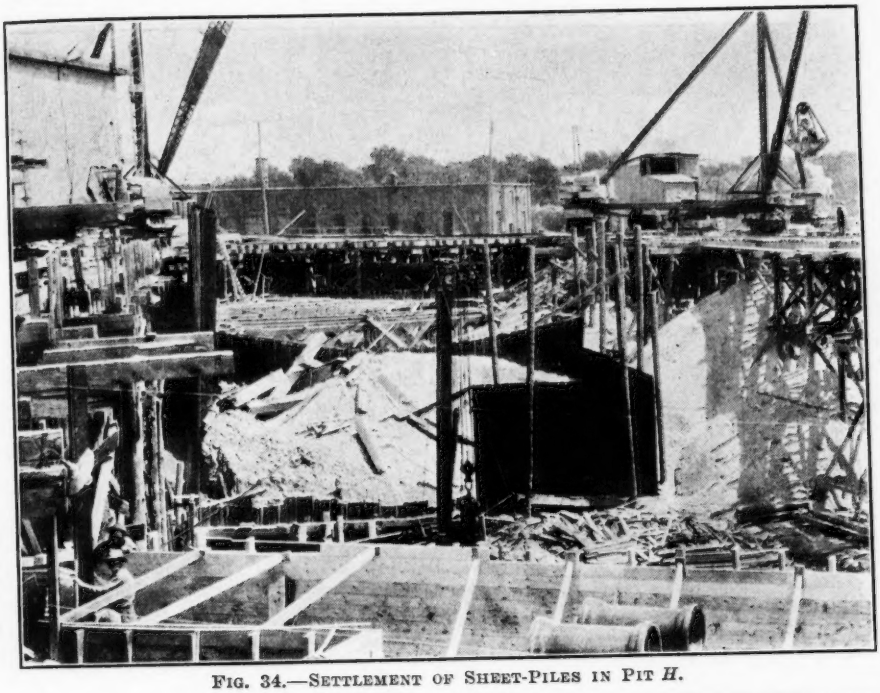

FIG. 34.—SETTLEMENT OF SHEET-PILES IN PIT *H*.

FIG. 35.—COMPLEX BRACING SYSTEM REQUIRED FOR PIT *N*.

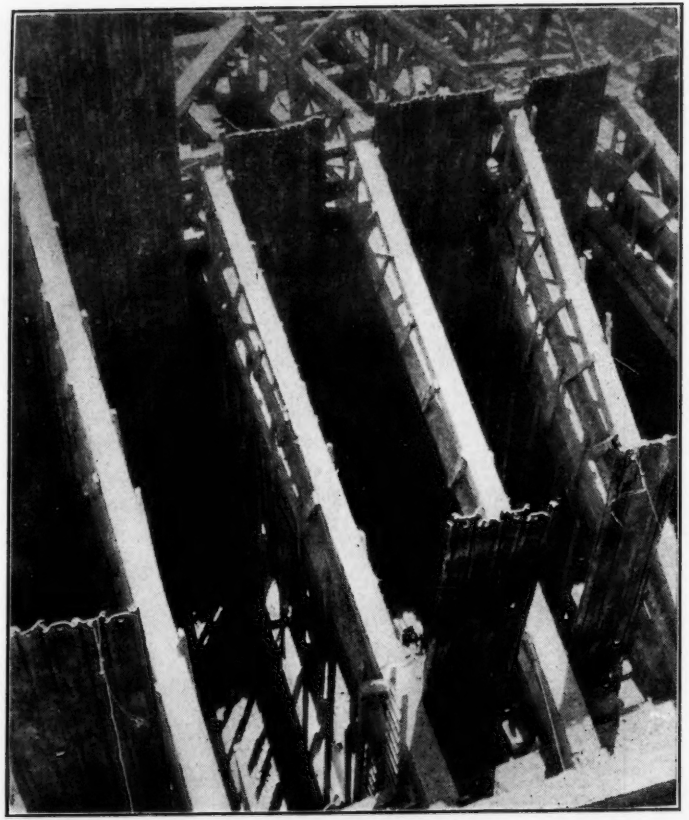

FIG. 36.—METHOD OF INTRODUCING THROUGH BRACING WITHOUT DISTURBING OTHER BRACING IN PLACE.

FIG. 37.—VIEW SHOWING COFFER-DAM CELLS IN PLACE WITH TIMBER BRACING REMOVED.

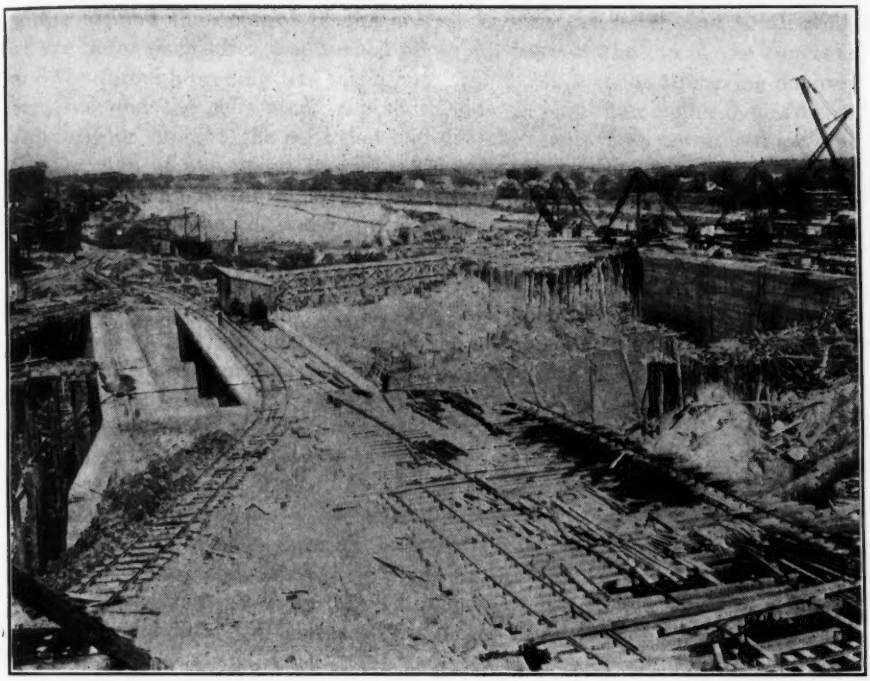

FIG. 38.—CHIPPAWA SIPHON CULVERT READY TO BE FLOODED, SEPTEMBER 10, 1928.

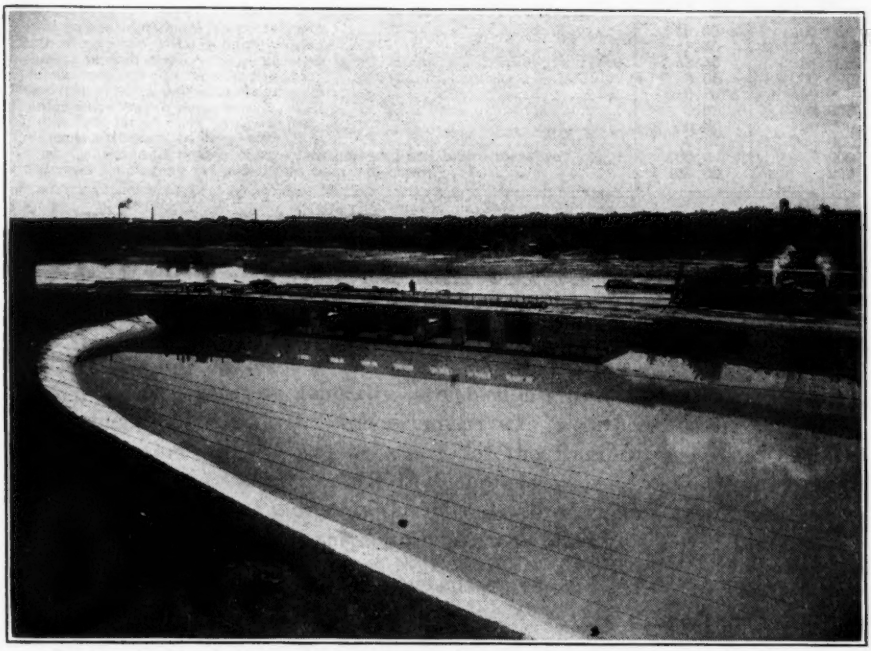

FIG. 39.—GENERAL VIEW OF CHIPPAWA SIPHON CULVERT LOOKING NORTHEAST. ENTRANCE CHANNEL IN FOREGROUND.

area of the culvert was then ready to be flooded. Accordingly, all steel sheet-piling around the exterior of the culvert area in the north and south walls of the outer enclosures was burned off at the level of the top of the concrete, and the through-bracing was taken out. With this interior timbering entirely removed and the cells along the south side of the outer coffer-dam free of any interior support, the cells that had deflected when first unwatered gave no further sign of movement (see Fig. 37). No doubt, this was due to the fact that the clay fill in the cells had become dry and thoroughly consolidated.

The cells of the coffer-dam in the prism north and south of the culvert were then removed, the culvert area was flooded, and the new Ship Canal channel established over the siphon culvert. Fig. 38 is a view showing the siphon culvert ready to be flooded. On December 7, 1928, just two years and ten months after the construction of the cellular coffer-dam was started, the first boat passed over the new structure. Fig. 39 is a view of the completed siphon culvert.

The excavation of the intake and discharge channels of the culvert was completed in 1929 and Chippawa Creek was diverted through the new structure on September 5, 1929. The north and south wing-walls of the siphon culvert were completed on December 6, 1929. The Second and Third Canal aqueducts were then isolated from the creek in the canal prism and that of the Third Canal was removed by dredging in 1931.

For the construction of the Chippawa Creek Siphon Culvert, the contractor was paid at the contract unit prices listed in Table 11.

TABLE 11.—DISTRIBUTION OF COSTS, CHIPPAWA CREEK SIPHON

Items	Unit cost	Quantity
Earth excavation (open), in cubic yards	$0.63	300 000
Earth excavation (pit), in cubic yards	1.50	170 000
Concrete in siphon culvert, in cubic yards	7.50	84 000
Steel sheet-piling (driven), in pounds	0.05	15 000 000
Removing steel sheet-piling, in pounds	0.0075	6 489 000
Reinforcing steel, in pounds	0.05	808 000
Timber foundation piles, in linear feet	0.80	196 000
Timber trestle piles, in linear feet	0.85	246 000
British Columbia fir timber bracing, in thousand feet board measure	100.0	4 000
Hardwood bracing, in thousand feet board measure	150.00	999

In all cases where steel sheet-piling, timber bracing, etc., were removed by the contractor, these materials became his property, If they were in good usable condition, their re-use in the work was permitted, and the contractor was paid the prevailing contract prices. The total cost of the siphon culvert structure, including materials, such as cement, supplied by the Department, was $3 800 000.

While the contractor generally developed his own construction methods, the problem of building the siphon culvert was so involved and embraced so many operations that it was necessary for the Engineering Department of the Welland Ship Canal to cover the complete cycle of operations in detail in the specification. Practically the only detail of this operation left to the contractor's discretion was the selection of the plant.

ACKNOWLEDGMENTS

The writer is particularly indebted to Mr. A. J. Grant, Engineer in Charge of the Welland Ship Canal, for his review of this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

STRESSES IN INCLINED ARCHES OF MULTIPLE-ARCH DAMS

BY GEORGE E. GOODALL[1] AND IVAN M. NELIDOV,[2]
ASSOCIATE MEMBERS, AM. SOC. C. E.

SYNOPSIS

The paper here presented contains the derivation of analytical formulas for moments, thrusts, and shears in the inclined circular arches of multiple-arch dams. All the equations were derived by integration through application of the theorem of Castigliano, and they include the internal elastic work due to shear.

As they are all necessarily long and cumbersome, coefficients for obtaining moments, thrusts, and shears were computed, and curves representing these coefficients are presented for the convenience of the designing engineer. The water load on the arch may be divided into two parts, the first being the uniform water load due to the head measured from the free water surface to the arch crown, and the second, the variable water load which results from the inclination of the arch. In addition to the water load, the dead weight of the arch and the temperature stresses must be considered, and the moments, thrusts, and shears of all these influences must be combined algebraically to determine maximum stresses.

INTRODUCTION

In the past, the moments, thrusts, and shears due to variable water load and dead load have been determined by graphical methods or by the analytical method developed by Dr. N. Kelen.[3] This latter method provides curves from which the crown thrust may be obtained directly, but does not

NOTE.—Written discussion on this paper will be closed in August, 1932, *Proceedings*.

[1] Senior Engr. of Hydr. Structure Design, Div. of Water Resources, State Dept. of Public Works, Sacramento, Calif.

[2] Senior Engr. of Hydr. Structure Design, State Dept. of Public Works, Sacramento, Calif.

[3] "Die Staumauern", von Dr.-Ing. N. Kelen, 1926, p. 44.

include curves or coefficients from which the abutment thrust, abutment shear, and bending moments may be directly obtained. Dr. Kelen's formulas omit the effect of the internal elastic work due to shear.

The late William Cain, M. Am. Soc. C. E., in his paper[4] entitled, "The Circular Arch Under Normal Loads," derived the necessary formulas for the moments, thrusts, and shears due to uniform water load and temperature changes. Professor Cain did not consider the problem of the non-uniform loads imposed on the inclined arches of multiple-arch dams, namely, the variable water load and the dead weight of the arch.

The derivations which follow are for the moments, thrusts, and shears due to the variable water load and the dead weight of the arch. All the formulas include the internal elastic work due to shear.

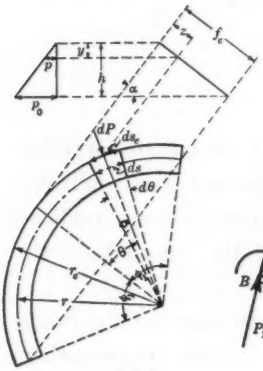

FIG. 1

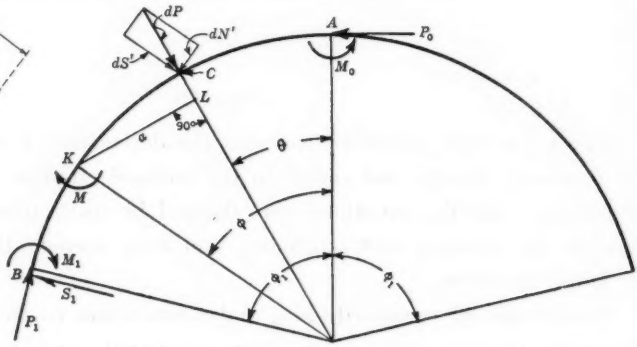

FIG. 2

MOMENTS, THRUSTS, AND SHEARS DUE TO VARIABLE WATER LOAD FOR ARCHES WITH FIXED ENDS

Fig. 1 represents a section of the arch taken normal to the generator of the arch barrel, and also the hydrostatic pressure diagram. The arch is considered as being fixed at the ends.

Let, t = uniform radial thickness of arch, in feet;
r = radius of center line of arch, in feet;
r_e = radius of extrados, in feet;
w = unit weight of water, in pounds per cubic foot;
θ = angle between the crown and any point;
ϕ_1 = one-half the central angle of the arch;
α = angle of inclination of arch generator with the horizontal;
y = vertical distance of any point on the arch below the crown;
h_o = vertical distance between the crown and the springing line of the arch;
p_o = pressure due to the head, h_o; and,
p = pressure due to the head, y.

In the derivation that follows, the only load acting on the arch is the water load which varies from zero at the crown to p_o at the springing line due to the inclination of the arch, α.

[4] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

From Fig. 1, $p_o = wh_o$; $p = p_o \frac{y}{h_o} = p_o \frac{z}{f_e}$; $f_e = \frac{h_o}{\cos \alpha}$; and, $z = r_e - r_e \cos \theta = r_e (1 - \cos \theta)$. Therefore,

$$p = p_o r_e \frac{(1 - \cos \theta)}{h_o} \cos \alpha = r_e (1 - \cos \theta) w \cos \alpha. \quad \ldots\ldots(1)$$

Since $w \cos \alpha = w'$ is a constant for any given inclination of the arch generator, Equation (1) may be written:

$$p = w' r_e (1 - \cos \theta) \quad \ldots\ldots(2)$$

From Fig. (2), $dp = p\ ds_e = w' r_e (1 - \cos \theta)\ ds_e$; and $\frac{de_e}{ds} = \frac{r_e}{r}$, or $ds_e = \frac{r_e}{r} ds$. Therefore,

$$dp = w' \frac{r_e^2}{r} (1 - \cos \theta)\ ds \quad \ldots\ldots(3)$$

but $ds = r d\theta$, and,

$$dp = w' r_e^2 (1 - \cos \theta)\ d\theta \quad \ldots\ldots(4)$$

Furthermore, $dM' = + a\ dP$; but $a = r \sin (\phi - \theta)$. Therefore,

$$dM' = + w' r_e^2 r (1 - \cos \theta) \sin (\phi - \theta)\ d\theta$$

and,

$$M' = w' r_e^2 r \int_0^{\phi} (1 - \cos \theta) \sin (\phi - \theta)\ d\theta$$

$$= w' r_e^2 r \left[\sin^2 \phi + \cos \phi (\cos \phi - 1) - \sin \phi (0.5 \sin \phi \cos \phi + 0.5 \phi) + 0.5 \cos \phi \sin^2 \phi \right]$$

or,

$$M' = w' r_e^2 r (1 - \cos \phi - 0.5 \phi \sin \phi) \quad \ldots\ldots(5)$$

Equation (5) gives the moment due to the external loads at the point, K. As each of the elementary loads, dP, is radial, it follows that the resultant of all these loads, P, between the crown and K must pass through the center of curvature of the arch. This resultant, ΣP, may be resolved into two components, the first being radial to K representing the shearing component of the external loads, and the second, tangential and representing the component of thrust, N'.

The force, N', is equal to M' divided by the radius, or,

$$N' = \frac{M'}{r} = w' r_e^2 (1 - \cos \phi - 0.5 \phi \sin \phi) \quad \ldots\ldots(6)$$

Taking moments about K of all the forces from the crown to K,

$$M = M_o + w' r_e^2 r (1 - \cos \phi - 0.5 \phi \sin \phi) - Pr (1 - \cos \phi) \quad \ldots\ldots(7)$$

$$P = P_o \cos \phi + w' r_e^2 (1 - \cos \phi - 0.5 \phi \sin \phi) \quad \ldots\ldots(8)$$

At any point, K (Fig. 2), the shearing component of an elementary load, dP, is the component parallel to OK, thus,

$$dS' = dP \cos(\phi - \theta) \quad \ldots\ldots (9)$$

Substituting dP from Equation (4), Equation (9) becomes,

$$dS' = w' r_e^2 (1 - \cos\theta) \cos(\phi - \theta)\, d\theta \quad \ldots\ldots (10)$$

The total shearing component of all external loads from A to K is,

$$S' = \int_0^{\phi} w' r_e^2 (1 - \cos\theta) \cos(\phi - \theta)\, d\theta$$

or,

$$S' = \frac{w' r_e^2}{2} (\sin\phi - \phi\cos\phi) \quad \ldots\ldots (11)$$

The total shear at K is equal to the sum of the components of all forces parallel to OK, between the crown and K, or,

$$S = \frac{w' r_e^2}{2} (\sin\phi - \phi\cos\phi) - P_o \sin\phi \quad \ldots\ldots (12)$$

The total internal elastic work for the semi-arch due to M, P, and S, is,

$$L = \frac{r}{2EI} \int_0^{\phi_1} M^2\, d\phi + \frac{r}{2Et} \int_0^{\phi_1} P^2\, d\phi + \frac{2.88\, r}{2Et} \int_0^{\phi_1} S^2\, d\phi \ldots (13)$$

Since the rotation of the crown section is zero, $\frac{\partial L}{\partial M_o} = 0$; also, since there is no displacement of the crown section in the direction of P_o, $\frac{\partial L}{\partial P_o} = 0$.

Taking partial derivatives of Equations (7), (8), (12), and (13), $\frac{\partial M}{\partial M_o} = 1$; $\frac{\partial M}{\partial P_o} = -r(1 - \cos\phi)$; $\frac{\partial P}{\partial M_o} = 0$; $\frac{\partial P}{\partial P_o} = \cos\phi$; $\frac{\partial S}{\partial M_o} = 0$; $\frac{\partial S}{\partial P_o} = -\sin\phi$; and $\frac{\partial L}{\partial M_o} = \frac{r}{EI} \int_0^{\phi_1} \frac{M\, \partial M\, d\phi}{\partial M_o} = 0$. Therefore,

$$\int_0^{\phi_1} M\, d\phi = 0 \quad \ldots\ldots (14)$$

By the aid of Equation (7) this reduces to,

$$\int_0^{\phi_1} [M_o + w' r_e^2 r (1 - \cos\phi - \frac{1}{2}\phi\sin\phi) - P_o r (1 - \cos\phi)]\, d\phi = 0$$

or,

$$M_o \phi_1 + w' r_e^2 r (\phi_1 - \frac{3}{2}\sin\phi_1 + \frac{1}{2}\phi_1\cos\phi_1) - P_o r (\phi_1 - \sin\phi_1) = 0 \ldots (15)$$

From $\frac{\partial L}{\partial P_o} = 0$,

$$\frac{r}{I} \int_0^{\phi_1} M \frac{\partial M}{\partial P_o}\, d\phi + \frac{r}{t} \int_0^{\phi_1} P \frac{\partial P}{\partial P_o}\, d\phi + \frac{2.88\, r}{t} \int_0^{\phi_1} S \frac{\partial S}{\partial P_o}\, d\phi = 0 \ldots (16)$$

Multiplying by $\frac{I}{r}$ and making $\frac{I}{t} = k^2$, this becomes,

$$-r\int_0^{\phi_1} M\,(1-\cos\phi)\,d\phi + k^2\int_0^{\phi_1} P\cos\phi\,d\phi + 2.88k^2\int_0^{\phi_1} S\,(-\sin\phi)\,d\phi = 0 \quad \ldots\ldots(17)$$

By Equation (14), $\int_0^{\phi_1} M\,d\phi = 0$, so the first term of Equation (17) becomes,

$$r\int_0^{\phi_1} M\cos\phi\,d\phi = r\int_0^{\phi_1}[M_0 + w'r_e^2 r\,(1-\cos\phi-\frac{1}{2}\phi\sin\phi) - P_0 r\,(1-\cos\phi)]\cos\phi\,d\phi \quad \ldots\ldots(18)$$

which reduces to the form,

$$r\int_0^{\phi_1} M\cos\phi\,d\phi = r\left[M_0\sin\phi_1 + w'r_e^2 r\left(\sin\phi_1 - \frac{5}{8}\sin\phi_1\cos\phi_1 - \frac{3\phi_1}{8} - \frac{\phi_1}{4}\sin^2\phi_1\right) - P_0 r\left(\sin\phi_1 - \frac{1}{2}\sin\phi_1\cos\phi_1 - \frac{\phi_1}{2}\right)\right] \ldots\ldots(19)$$

Likewise, the second term of Equation (17) becomes,

$$k^2\int_0^{\phi_1} P\cos\phi\,d\phi = k^2\int_0^{\phi_1}\left[P_0\cos\phi + w'r_e^2\left(1-\cos\phi-\frac{\phi}{2}\sin\phi\right)\right]\cos\phi\,d\phi \quad \ldots(20)$$

or,

$$k^2\int_0^{\phi_1} P\cos\phi\,d\phi = k^2\left[P_0\left(\frac{1}{2}\sin\phi_1\cos\phi_1 + \frac{\phi_1}{2}\right) + w'r_e^2\left(\sin\phi_1 - \frac{5}{8}\sin\phi_1\cos\phi_1 - \frac{3\phi_1}{8} - \frac{\phi_1}{4}\sin^2\phi_1\right)\right] \ldots\ldots(21)$$

Finally, the third term of Equation (17) becomes,

$$-2.88k^2\int_0^{\phi_1} S\sin\phi\,d\phi = -2.88k^2\int_0^{\phi_1}\left[\frac{w'r_e^2}{2}(\sin\phi-\phi\cos\phi) - P_0\sin\phi\right]\sin\phi\,d\phi \quad \ldots(22)$$

or,

$$-2.88k^2\int_0^{\phi_1} S\sin\phi\,d\phi = -2.88k^2\left[\frac{w'r_e^2}{4}\left(-\frac{3}{4}\sin 2\phi_1 + \phi_1 + \frac{\phi_1\cos 2\phi_1}{2}\right) - P_0\left(\frac{\phi_1}{2} - \frac{\sin 2\phi_1}{4}\right)\right] \ldots\ldots(23)$$

After Equations (19), (21), and (23) have been substituted in Equation (17), M_0 may be eliminated by multiplying through by ϕ_1, and subtracting the result from Equation (15), multiplied by $r\sin\phi_1$.

The next step is to replace $\sin \phi_1$, $\cos \phi_1$, by its equivalent, $\frac{1}{2} \sin 2\phi_1$, and solve for P_o. The final result is:

$$P_o = w' r_e^2 \left\{ \left(2 \sin \phi_1 - \frac{3 \sin^2 \phi_1}{\phi} + \frac{\sin 2\phi_1}{2} \right) - \left(\frac{1 + k^2}{r^2} \right) \left(2 \sin \phi_1 - \frac{5}{8} \sin 2\phi_1 - \phi_1 + \frac{\phi_1 \cos 2\phi_1}{4} \right) + 5.76 \frac{k^2}{r_2} \left[\frac{1}{4} \left(- \frac{3}{4} \sin 2\phi_1 + \phi_1 + \frac{\phi_1 \cos 2\phi_1}{2} \right) \right] \right\} \div \left\{ \left(1 + \frac{k^2}{r^2} \right) \left(\phi_1 + \frac{\sin 2\phi_1}{2} \right) - \frac{2 \sin^2 \phi_1}{\phi_1} + 2.88 \frac{k^2}{r^2} \left(\frac{\phi_1 - \sin 2\phi_1}{2} \right) \right\} \quad \ldots\ldots\ldots\ldots (24)$$

The denominator of Equation (24) checks Professor Cain's expression for D_s divided by ϕ_1.[5]

Since w' is the unit weight of water multiplied by the cosine of the angle of inclination of the arch, and r_e and $2\phi_1$, for a given dam, are known, Equation (24) for the crown thrust may be written,

$$P_o = w' r_e^2 \, Z \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

in which, Z embraces terms containing only $\frac{t}{r}$ and functions of the central angle, $2\phi_1$. For the convenience of the designing engineer, numerous values of the coefficient, Z, have been calculated for all values of $\frac{t}{r}$ and $2\phi_1$ likely

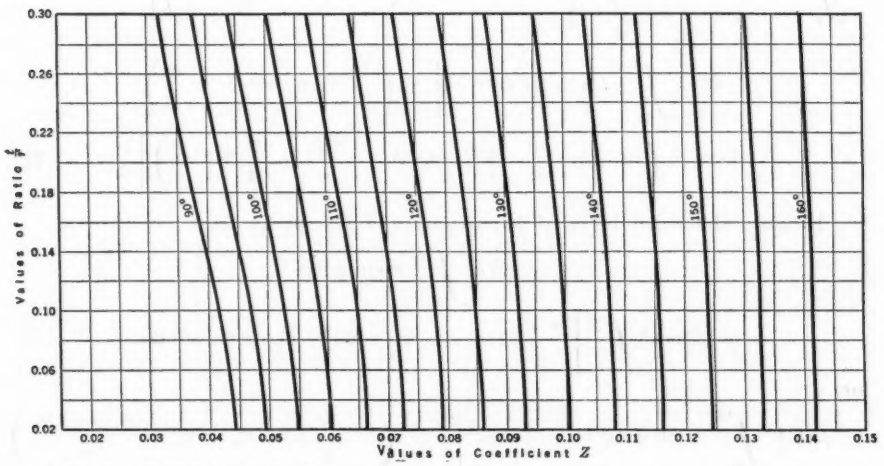

FIG. 3.—CROWN THRUST DUE TO VARIABLE WATER LOAD ON MULTIPLE-ARCH DAMS, $P_o = w' r_e^2 Z$

to be met in the design of multiple-arch dams and, from these coefficients, curves have been plotted so that the crown thrust may readily be computed. The values of the coefficient, Z, are plotted in Fig. 3.

[5] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 237 and 269.

If ϕ_1 is substituted for ϕ in Equation (8), the thrust at the abutment may be written in terms of Z, as follows:

$$P_1 = w' r_e^2 \left[Z \cos \phi_1 + \left(1 - \cos \phi_1 - \frac{1}{2} \phi_1 \sin \phi_1 \right) \right] = w' r_e^2 Z_1 \ldots (26)$$

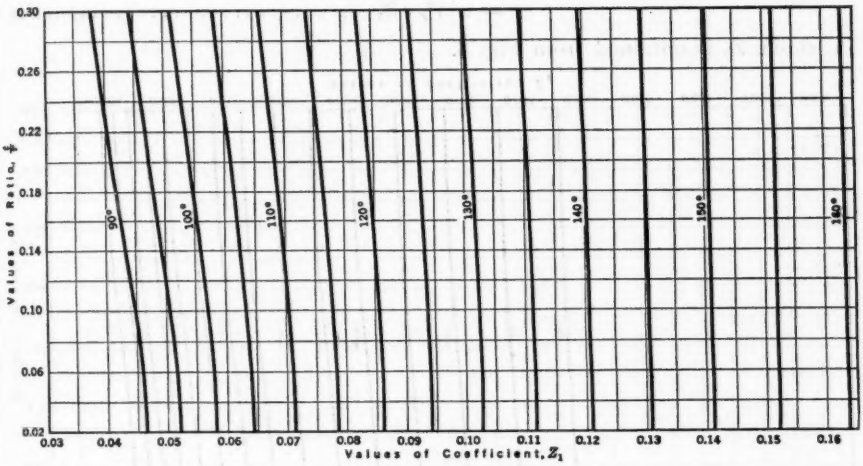

FIG. 4.—ABUTMENT THRUST DUE TO VARIABLE WATER LOAD ON MULTIPLE-ARCH DAMS, $P_1 = w' r_e^2 Z_1$

in which, the value of Z_1 may be obtained from Fig. 4. Then, from Equation (15), the crown moment M_o is:

$$M_o = w' r_e^2 r \left[\left(1 - \frac{\sin \phi_1}{\phi_1} \right) Z - \left(1 - \frac{3 \sin \phi_1}{2 \phi_1} + \frac{1}{2} \cos \phi_1 \right) \right] = w' r_e^2 r Z_2 \ldots (27)$$

in which, the value of Z_2 is obtained from Fig. 5.

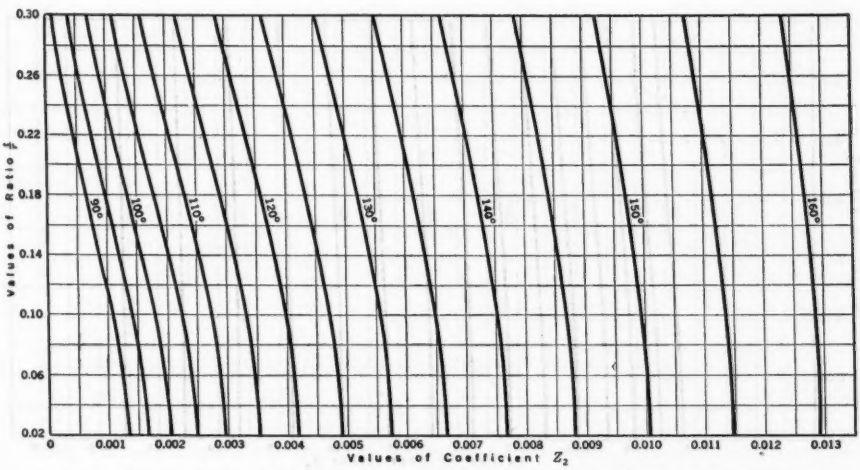

FIG. 5.—CROWN MOMENT DUE TO VARIABLE WATER LOAD ON MULTIPLE-ARCH DAMS, $M_o = w' r_e^2 r Z_2$

The moment at the abutment is obtained by combining Equations (7) and (25), with the following result:

$$M_1 = -w' r_e^2 r \left[Z \left(\frac{\sin \phi_1}{\phi_1} - \cos \phi_1 \right) + \frac{3}{2} \left(\cos \phi_1 + \frac{\phi_1 \sin \phi_1}{3} - \frac{\sin \phi_1}{\phi_1} \right) \right]$$
$$= w' r_e^2 r Z_3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

in which, Z_3 is obtained from Fig. 6.

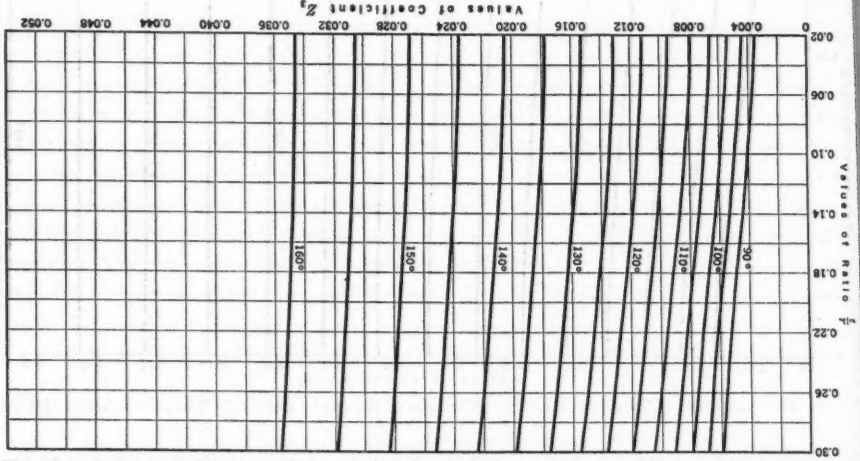

FIG. 6.—ABUTMENT MOMENT DUE TO VARIABLE WATER LOAD ON MULTIPLE-ARCH DAMS, $M_1 = w' r_e^2 r Z_3$

From Equation (12), the shear at the abutment is equal to,

$$S_1 = w' r_e^3 \left[\tfrac{1}{2} (\sin \phi_1 - \phi_1 \cos \phi_1) - Z \sin \phi_1 \right] = w' r_e^2 Z_4 \ldots\ldots\ldots (29)$$

in which, Z_4 is obtained from Fig. 7.

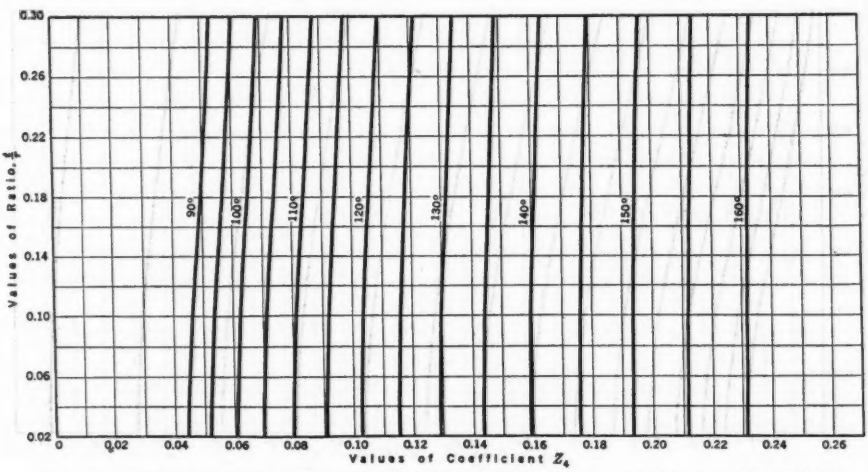

FIG. 7.—ABUTMENT SHEAR DUE TO VARIABLE WATER LOAD ON MULTIPLE-ARCH DAMS, $S_1 = w_1 r_e^2 Z_4$

Moments, Thrusts, and Shears Due to Weight of Arch for Arches with Fixed Ends

In determining the stresses due to the weight of the arch which is considered as fixed at the abutments, the only forces to be considered are the components of the weight of the elementary lengths, ds, normal to the generator of the arch barrel acting parallel to the plane of the buttresses

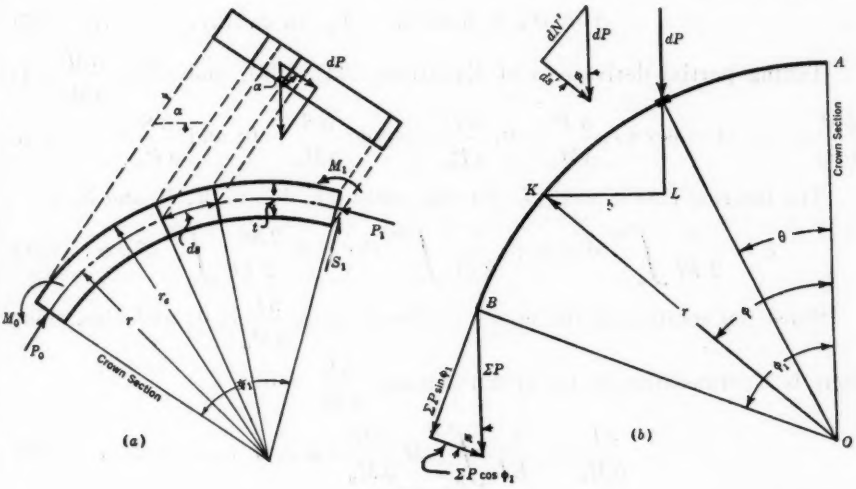

FIG. 8

(Fig. 8). The weight of an elementary length of arch, ds (for an arch slice of 1 ft. slant height) is $w_c t\, ds$, in which, w_c = unit weight of concrete, in pounds per cubic foot.

The down-stream component of weight of the elementary length is, $dP = w_c t\, ds \cos \alpha = w'_c\, t\, ds$, in which, $w'_c = w \cos \alpha$, is a constant.

The bending moment at K (Fig. 8) due to dP is, $dM' = a\, dP$; but $a = r\,(\sin \phi - \sin \theta)$; $dP = w'_c t\, ds$; and, $ds = r d\theta$. Therefore,

$$dM' = w'_c\, tr^2\,(\sin \phi - \sin \theta)\, d\,\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

and,

$$M' = \int_0^{\phi} w'_c\, tr^2\,(\sin \phi - \sin \theta)\, d\,\theta \ldots\ldots\ldots\ldots (31)$$

or,

$$M' = w'_c\, tr^2\,(\phi \sin \phi + \cos \phi - 1) \ldots\ldots\ldots\ldots (32)$$

Taking moments about K of all the forces from the crown to K:

$$M = M_o + w'_c\, tr^2\,(\phi \sin \phi + \cos \phi - 1) - P_o r\,(1 - \cos \phi) \ldots . (33)$$

The thrust, P, at K, is equal to the sum of the components parallel to the tangent at K of all the forces between the crown and K, or,

$$P = P_o \cos \phi + \Sigma\, P \sin \phi \ldots\ldots\ldots\ldots\ldots\ldots (34)$$

Furthermore,

$$\Sigma P = \int_0^{\phi} w'_c\, t\, ds = \int_0^{\phi} w'_c\, tr\, d\theta = w'_c\, tr\, \phi \quad \ldots\ldots\ldots(35)$$

and hence Equation (34) becomes,

$$P = P_0 \cos\phi + w'_c\, tr\, \phi \sin\phi \ldots\ldots\ldots\ldots\ldots(36)$$

The shear is equal to,

$$S = w'_c\, tr\, \phi \cos\phi - P_0 \sin\phi \ldots\ldots\ldots\ldots\ldots(37)$$

Taking partial derivatives of Equations (33), (36), and (37): $\frac{\partial M}{\partial M_0} = 1$; $\frac{\partial M}{\partial P_0} = -r(1-\cos\phi)$; $\frac{\partial P}{\partial M_0} = 0$; $\frac{\partial P}{\partial P_0} = \cos\phi$; $\frac{\partial S}{\partial M_0} = 0$; and $\frac{\partial S}{\partial P_0} = -\sin\phi$.

The internal elastic work, L, for the semi-arch, due to M, P, and S, is:

$$L = \frac{r}{2EI}\int_0^{\phi_1} M^2\, d\phi + \frac{r}{2Et}\int_0^{\phi_1} P^2\, d\phi + \frac{2.88\, r}{2Et}\int_0^{\phi_1} S^2\, d\phi \ldots(38)$$

Since the rotation of the crown section is zero, $\frac{\partial L}{\partial M_0} = 0$; and, also, since there is no translation of the crown section, $\frac{\partial L}{\partial P_0} = 0$:

$$\frac{\partial L}{\partial M_0} = \frac{r}{EI}\int_0^{\phi_1} M \frac{\partial M}{\partial M_0}\, d\phi = 0 \quad \ldots\ldots\ldots\ldots\ldots(39)$$

therefore,

$$\int_0^{\phi_1} M\, d\phi = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots(40)$$

By the aid of Equation (33), this becomes:

$$\int_0^{\phi_1} \left[M_0 + w'_c\, tr\, (\phi\sin\phi + \cos\phi - 1) - P_0(r - r\cos\phi) \right] d\phi = 0$$

$$\left[M_0\phi + w'_c\, tr\, (\sin\phi - \phi\cos\phi + \sin\phi - \phi) - P_0 r\, (\phi - \sin\phi) \right]_0^{\phi_1} = 0$$

or,

$$M_0\phi_1 + w'_c\, tr\, (2\sin\phi_1 - \phi_1\cos\phi_1 + \phi_1) - P_0 r\, (\phi_1 - \sin\phi_1) = 0 \ldots(41)$$

From $\frac{\partial L}{\partial P_0} = 0$:

$$\frac{r}{I}\int_0^{\phi_1} M \frac{\partial M}{\partial P_0}\, d\phi + \frac{r}{t}\int_0^{\phi_1} P \frac{\partial P}{\partial P_0}\, d\phi + \frac{2.88\, r}{t}\int_0^{\phi_1} S \frac{\partial S}{\partial P_0}\, d\phi = 0 \ldots(42)$$

Multiplying by $\frac{I}{r}$ and making $\frac{I}{t} = k^2$, Equation (42) becomes:

$$-r\int_0^{\phi_1} M(1-\cos\phi)\, d\phi + k^2\int_0^{\phi_1} P\cos\phi\, d\phi + 2.88k^2\int_0^{\phi_1} S(-\sin\phi)\, d\phi = 0 \quad \ldots\ldots\ldots\ldots(43)$$

By Equation (40) the first term of Equation (43) becomes:

$$r\int_0^{\phi_1} M\cos\phi\,d\phi = r\int_0^{\phi_1}\Big[M_0 + w'_c\,tr^2\,(\phi\sin\phi+\cos\phi-1) - P_0 r\,(1-\cos\phi)\Big]\cos\phi\,d\phi \quad\ldots\ldots\ldots\ldots\ldots\ldots(44)$$

which returns to the form:

$$r\int_0^{\phi_1} M\cos\phi\,d\phi = r\left[M_0\sin\phi_1 + w'_c tr^2\left(\frac{3}{8}\sin 2\phi_1 - \frac{\phi_1\cos 2\phi_1}{4} + \frac{\phi_1}{2}\right) - \sin\phi_1 - P_0 r\left(\sin\phi_1 - \frac{\sin 2\phi_1}{4} - \frac{\phi_1}{2}\right)\right]\ldots\ldots\ldots\ldots(45)$$

Likewise, the second term of Equation (43) becomes:

$$k^2\int_0^{\phi_1} P\cos\phi\,d\phi = k^2\int_0^{\phi_1}\Big[P_0\cos\phi + w'_c\,tr\,\phi\sin\phi\Big]\cos\phi\,d\phi\ldots(46)$$

which reduces to the form,

$$k^2\int_0^{\phi_1} P\cos\phi\,d\phi = k^2\left[P_0\left(\frac{1}{4}\sin 2\phi_1 + \frac{1}{2}\phi_1\right) + w'_c\,\frac{tr}{4}\left(\frac{1}{2}\sin 2\phi - \phi_1\cos 2\phi_1\right)\right]\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(47)$$

Finally, the third term of Equation (43) becomes:

$$-2.88k^2\int_0^{\phi_1} S\sin\phi\,d\phi = -2.88k^2\int_0^{\phi_1}\Big[w'_c\,tr\,\phi\cos\phi - P_0\sin\phi\Big]\sin\phi\,d\phi\quad\ldots\ldots(48)$$

or,

$$-2.88k^2\int_0^{\phi_1}\sin\phi\,d\phi = -2.88k^2\left[\frac{1}{4}w'_c\,tr\left(\frac{1}{2}\sin 2\phi_1 - \phi_1\cos 2\phi_1\right) - P_0\left(\frac{1}{2}\phi_1 - \frac{1}{4}\sin 2\phi_1\right)\right]\ldots\ldots\ldots\ldots\ldots\ldots(49)$$

After Equations (45), (47), and (49) have been substituted in Equation (43), M_0 may be eliminated by multiplying through by ϕ_1, and subtracting the result from Equation (41) multiplied by $r\sin\phi_1$. The next step is to solve for P_0 and simplify by multiplying both numerator and denominator by $\frac{2}{\phi_1 r^2}$. The final result is:

$$P_0 = w'_c tr\,\frac{\left(\frac{4\sin^2\phi_1}{\phi_1} - \frac{7}{4}\sin 2\phi_1 + \frac{\phi_1\cos 2\phi_1}{2} - \phi_1\right) + 1.88\frac{k^2}{r^2}\left(\frac{1}{4}\sin 2\phi_1 - \frac{\phi_1\cos 2\phi_1}{2}\right)}{\left(1+\frac{k^2}{r^2}\right)\left(\phi_1 + \frac{\sin 2\phi_1}{2}\right) - \frac{2\sin^2\phi_1}{\phi_1} + 2.88\,\frac{k^2}{r^2}\left(\phi_1 - \frac{\sin 2\phi_1}{2}\right)}. \quad (50)$$

Equation (50) may be written:

$$P_o = w'_c\, tr\, Z' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

By means of Equations (36) and (51), the formula for abutment thrust is:

$$P_1 = w'_c\, tr\, [Z' \cos \phi_1 + \phi_1 \sin \phi_1] = w'_c\, tr\, Z'_1 \ldots\ldots\ldots (52)$$

Equation (41), which gives the moment at the crown, may be written:

$$M_o = w'_c\, tr^2 \left[Z' \left(1 - \frac{\sin \phi_1}{\phi_1} \right) - \left(\frac{2 \sin \phi_1}{\phi_1} - \cos \phi_1 - 1 \right) \right] = w'_c\, tr\, Z'_2 \ldots (53)$$

From Equations (33), (51), and (53), the bending moment at the abutment now becomes:

$$M_1 = - w'_c\, tr^2 \left[Z' \left(\frac{\sin \phi_1}{\phi_1} - \cos \phi_1 \right) + \left(\frac{2 \sin \phi_1}{\phi_1} - 2 \cos \phi_1 - \phi_1 \sin \phi_1 \right) \right]$$
$$= w'_c\, tr^2\, Z'_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54)$$

The shear at the abutment is, from Equation (37),

$$S_1 = w'_c\, tr\, (\phi_1 \cos \phi_1 - Z' \sin \phi_1) = w'_c\, tr\, Z'_4 \ldots\ldots\ldots (55)$$

Moments, Thrusts, and Shears Due to Uniform Water Load for Arches with Fixed Ends

In the preceding part of this paper, equations were derived for the moments, thrusts, and shears due to the variable water load and the dead load of the arch itself. The formulas derived by the late Professor Cain, in his paper,[6] "The Circular Arch Under Normal Loads," have been used as a basis for curves similar to Figs. 3 to 7, inclusive, which serve as a convenient means of obtaining the stresses due to the uniform water load. As in the foregoing remarks, the formulas are those which include the effect of shear.

According to Professor Cain's Equation (10), $(pr - P_o) = \dfrac{p r\, 2 \phi_1 \sin \phi_1 k^2}{D_s r^2}$;

but $pr = p_e r_e$ and $r_e = r + \dfrac{1}{2t} = r \left(1 + \dfrac{t}{2r} \right)$; also $k^2 = \dfrac{t^2}{12}$. Therefore,

$$pr - P_o = p_e r \left(1 + \frac{t}{2r} \right) \frac{2 \phi_1 \sin \phi_1\, t^2}{12\, D_s\, r^2} = p_e r\, Q_o \ldots\ldots\ldots (56)$$

in which, $Q_o = \left(1 + \dfrac{t}{2r} \right) \dfrac{2 \phi_1 \sin \phi_1\, t^2}{12\, D_s\, r^2}$.

All the equations for moments and thrusts at the crown and abutments, as given by Professor Cain, may be expressed in terms of the coefficient, Q_o, as follows:

The thrust at the crown is:

$$P_o = \left[\left(1 + \frac{t}{2r} \right) - Q_o \right] p_e\, r \ldots\ldots\ldots\ldots\ldots (57)$$

[6] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 237 and 269.

in which, the coefficient of $p_e r$ is obtained from specially prepared curves similar to Figs. 3 to 7, inclusive.

The abutment thrust is:

$$P_1 = p_e r \left[\left(1 + \frac{t}{2r} \right) - Q_0 \cos \phi_1 \right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (58)$$

The moment at the crown section is:

$$M_0 = - p_e r^2 \left(1 - \frac{\sin \phi_1}{\phi_1} \right) Q_0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (59)$$

The moment at the abutment is:

$$M_1 = p_e r^2 \left(\frac{\sin \phi_1}{\phi_1} - \cos \phi_1 \right) Q_0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (60)$$

The shear at the abutment is:

$$S_1 = p_e r \, Q_0 \cos \phi_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (61)$$

Temperature Stresses

The crown thrust for the arch with fixed ends as given by Professor Cain is $H = \varepsilon \Delta \, tEI \, \frac{2 \phi_1 \sin \phi_1}{r^2 D_s}$, which may also be written:

$$H = \epsilon \Delta \, t \, Et \, \frac{2 \phi_1 \sin \phi_1 t^2}{12 D_s r^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (62)$$

in which, $\frac{2 \phi_1 \sin \phi_1 t^2}{12 D_s r^2}$ is obtained from specially prepared curves.

The abutment thrust is equal to:

$$H_1 = H \cos \phi_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (63)$$

and the shear is:

$$S_1 = H \sin \phi_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (64)$$

For a fall in temperature, the thrusts are negative and the shear is positive, whereas for a rise in temperature the signs are reversed.

The moment at the crown is:

$$M_0 = - Hr \left(1 - \frac{\sin \phi_1}{\phi_1} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (65)$$

and the moment at the abutment is:

$$M_1 = Hr \left(\frac{\sin \phi_1}{\phi_1} - \cos \phi_1 \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (66)$$

For a fall in temperature the signs of the moments are as shown, but for a rise in temperature, the signs are reversed. For convenience in making numerical computations, values of $\left(\frac{1 - \sin \phi_1}{\phi_1} \right)$ and $\left(\frac{\sin \phi_1}{\phi_1} - \cos \phi_1 \right)$ are read from the specially prepared curves previously mentioned.

Moments, Thrusts, and Shears Due to Variable Water Load for Arches with Hinged Ends

The following derivation is similar to that for the arch with "fixed ends" up to the application of the partial derivatives of the "Work Equation" (13). As before, there is no translation of the crown section in the direction of P_o due to P_o and $\frac{\partial L}{\partial P_o} = 0$.

The general formula for moment (Equation (7)), becomes equal to zero at the abutment when ϕ_1 is substituted for ϕ; thus,

$$M_o = P_o r\,(1 - \cos\phi_1) - w' r_e^2\, r\,(1 - \cos\phi_1 - \frac{\phi_1}{2}\sin\phi_1) \ldots\ldots (67)$$

Substituting this value of M_o in the general moment equation (Equation (7)),

$$M = P_o r\,(\cos\phi - \cos\phi_1) + w' r_e^2\, r(\cos\phi_1 + \frac{\phi_1}{2}\sin\phi_1 - \cos\phi - \frac{\phi}{2}\sin\phi_1) \ldots (68)$$

Taking partial derivatives, as before, of the equations for moment, thrust, and shear, with respect to P_o: $\frac{\partial M}{\partial P_o} = r\,(\cos\phi - \cos\phi_1)$; $\frac{\partial P}{\partial P_o} = \cos\phi$; $\frac{\partial S}{\partial P_o} = -\sin\phi$; and,

$$\frac{\partial L}{\partial P_o} = r\int_o^{\phi_1} M\,(\cos\phi - \cos\phi_1)\,d\phi + k^2\int_o^{\phi_1} P\cos\phi\,d\phi - 2.88k^2\int_o^{\phi_1} S\sin\phi\,d\phi = 0 \quad \ldots\ldots\ldots\ldots\ldots (69)$$

Integrating and solving for P_o:

$$P_o = w' r_e^2 \Bigg[2\,\phi_1 - \frac{15\sin 2\phi_1}{8} + \frac{7\,\phi_1\cos 2\phi_1}{4} + \frac{\phi_1^2\sin 2\phi_1}{2} - \frac{k^2}{r^2}\left(2\sin\phi_1 - \phi_1 - \frac{5\sin 2\phi_1}{8} + \frac{\phi_1\cos 2\phi_1}{4}\right) + 2.88\,\frac{k^2}{r^2}\left(\frac{\phi_1}{2} - \frac{3\sin 2\phi_1}{8} + \frac{\phi_1\cos 2\phi_1}{4}\right)\Bigg] \div \Bigg[\phi_1\,(2 + \cos 2\phi_1) - \frac{3\sin 2\phi_1}{2} + \frac{k^2}{r^2}\left(\phi_1 + \frac{\sin 2\phi_1}{2}\right) + 2.88\,\frac{k^2}{r^2}\left(\phi_1 - \frac{\sin 2\phi_1}{2}\right)\Bigg] \ldots\ldots\ldots (70)$$

The abutment thrust (P_1), crown moment (M_o), and abutment shear (S_1), may be obtained from Equations (8), (67) and (12), respectively, by introducing the value of P_o obtained from Equation (70) and by replacing ϕ by ϕ_1 in Equations (8) and (12).

Moments, Thrusts, and Shears, Due to Weight of Arch for Arches with Hinged Ends

Similar to the derivation of formulas for variable water load for arches with hinged ends, the general expression for moment is:

$$M = P_o r (\cos\phi - \cos\phi_1) + w'_c tr^2 (\phi \sin\phi + \cos\phi - \phi_1 \sin\phi_1 - \cos\phi_1) \quad ..(71)$$

The partial derivatives to be substituted in the work equation, $\left(\frac{\partial L}{\partial P_o} = 0\right)$, are: $\frac{\partial M}{\partial P_o} = r(\cos\phi - \cos\phi_1)$; $\frac{\partial P}{\partial P_o} = \cos\phi$; and $\frac{\partial S}{\partial P_o} = -\sin\phi$. The crown thrust is:

$$P_o = w'_c tr \frac{\frac{9}{4}\sin 2\phi_1 - \phi_1 (\frac{5}{2}\cos 2\phi_1 + 2 + \phi_1 \sin 2\phi_1) + 1.88\frac{k^2}{r^2}\left(\frac{\sin 2\phi_1}{4} - \frac{\phi_1 \cos 2\phi_1}{2}\right)}{\phi_1 (2 + \cos 2\phi_1) - \frac{3}{2}\sin 2\phi_1 + \frac{k^2}{r^2}\left(\phi_1 + \frac{\sin 2\phi_1}{2}\right) + 2.88\frac{k^2}{r^2}\left(\phi_1 - \frac{\sin 2\phi_1}{2}\right)} \quad .(72)$$

The abutment thrust and shear and the crown moment are then obtained by the same method as for variable water load.

Moments, Thrusts, and Shears for Uniform Water Load and Temperature for Arches with Hinged Ends

Formulas for moments, thrusts, and shears, due to these influences are the same as those derived by Professor Cain,[7] except that the influence of shear is included; thus,

$$(pr - P_o) = \frac{(2\, pr \sin\phi_1)\frac{k^2}{r^2}}{B_s} \quad .....(73)$$

and,

$$H = \frac{EI}{r^2 B_s} 2\varepsilon \Delta t \sin\phi_1 \quad .....(74)$$

in which,

$$B_s = \phi_1 (2 + \cos 2\phi_1) - \frac{3}{2}\sin 2\phi_1 + \frac{k^2}{r^2}\left(\phi_1 + \frac{\sin 2\phi_1}{2}\right) + 2.88\frac{k^2}{r^2}\left(\phi_1 - \frac{\sin 2\phi_1}{2}\right)$$

Conclusions

In this paper it has been possible only to present a thorough description of the method of analysis proposed, with selected sections of the curves that have been prepared. These should serve as a basis of discussion. The original paper, with a complete set of the curves, has been placed in the Engineering Societies Library.[8]

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[7] "The Circular Arch Under Normal Loads", by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 242 and 247.

[8] Address The Director, Engineering Societies Library, 33 West 39th Street, New York, N. Y., directly, for photostatic copies.

By reference to Table 1 (Appendix) certain definite conclusions may be drawn as to the importance of the various influences considered. It will be seen that the effect of the non-uniform loads (variable water load and weight of arch) is of increasing importance as the arch span is increased. This is especially true in the upper part of the dam where the bending moments due to the non-uniform loads are several times as large as those due to the uniform water load alone. In recent years the trend of design of multiple-arch dams has been toward an increase in arch span lengths. Span lengths of 40 ft. or more, are not uncommon in recent practice.

It will also be seen that reinforcing steel is necessary in both faces of the arch, as the sign of the summation of bending moments at a given section may be either positive or negative, and the arch should be reinforced for both conditions. In the past, some multiple-arch dams have been designed with reinforcing steel in one face of the arch only.

Maximum compressive stresses in the concrete will occur, in general, for full reservoir and changes in temperature. Maximum tensile stresses in the reinforcing steel may be expected when the reservoir is empty, accompanied also by changes in temperature.

As has been shown by the "Numerical Example" in the Appendix, the labor of determining the stresses in the inclined arches of multiple-arch dams by the formulas herein derived and by the aid of the diagrams proposed, is reduced to a minimum.

Acknowledgments

The writers wish to express their appreciation of the work of D. S. Cleavinger, Jun. Am. Soc. C. E., in plotting the many diagrams and that of Harold E. Von Bergen, Jun. Am. Soc. C. E., who checked the algebraic work of deriving the formulas.

APPENDIX

In order to demonstrate the use of the various curves and the effects of the various influences, the moments, thrusts, and shears for two typical multiple-arch dams of considerably different dimensions are given.

Numerical Example.—The first, which will be called Dam No. 1, has a clear spacing of buttresses of 40 ft. and a constant radius at the intrados of 24.5 ft. The increase in thickness of the arch rings is obtained by increasing the radius of the extrados from 26.5 ft. at the crest to 30.0 ft. at the base, with thicknesses of arch ranging from 2 to 5.5 ft. in a total height of dam of 168 ft. The central angles of the arches vary from 143 to 146 degrees. The calculations are for an arch ring of 1 ft. slant height, and the elevations given refer to the elevation of the inclined arch at its springing line. The generator of the arch barrel makes an angle of 47° with the horizontal. High-water surface is at Elevation 208. The weight of concrete is taken as 150 lb. per cu. ft., and the weight of water at 62.5 lb. per cu. ft. One set of calculations will be shown for Elevation 140 to illustrate the method, and the moments, thrusts, and shears for the entire arch will be tabulated.

The head at the arch crown is: $(208 - 140) - 27.08 \cos 47° \sinh 71° = 45'$ $= 55.32$, and $p_e = 55.32 \times 62.5 = 3\,460$ lb. per sq. ft., in which, 27.08 is the radius of the extrados.

The thickness of the arch is 2.58 ft., making the mean radius, r, equal 25.79 ft. and $\frac{t}{r}$, 0.10. The value of $p_e r = 89\,150$; and that of $p_e r^2 = 2\,300\,000$.

Figs. 9(*a*), 9(*b*), 9(*c*), 9(*d*), and 9(*e*) are selected sections of the computation curves, from which the values of Q_o, $\left[(1 + \frac{t}{2r}) - Q_o\right]$; $\left[(1 + \frac{t}{2r}) - Q_o \cos \phi_1\right]$; $Q_o (1 - \frac{\sin \phi_1}{\phi_1})$; and, $Q_o (\frac{\sin \phi_1}{\phi_1} - \cos \phi_1)$, respectively, are obtained for $\frac{t}{r} = 0.10$ and $2\,\phi_1 = 143.5$ degrees.

The moments, thrusts, and shears for uniform water load are, therefore:

P_o (from Equation (57) and Fig. 9(*b*)) $= 89\,150 \times 1.035 = 92\,300$ lb;

P_1 (from Equation (58) and Fig. 9(*c*)) $= 89\,150 \times 1.045 = 93\,200$ lb.;

S^1 (from Equation (61) and Fig. 9 (*a*)) $= 89\,150 \times 0.015 \times 0.9497 = 1\,270$ lb.;

M_o (from Equation (59) and Fig. 9 (*d*)) $= -2\,300\,000 \times 0.00356 = -8\,185$ ft-lb.; and,

M_1 (from Equation (60) and Fig. 9 (*e*)) $= 2\,300\,000 \times 0.0065 = 14\,940$ ft-lb.

The calculations for variable water load at Elevation 140 are, as follows: $w' = 62.5 \times \cos 47° = 42.6$ lb.; $w' r^2_e = 42.6 \times 27.08^2 = 31\,250$; and $w' r^2_e\, r = 31\,250 \times 25.79 = 806\,300$.

For $\frac{t}{r} = 0.10$ and $2\,\phi_1 = 143.5°$, the values of the coefficients, Z, Z_1, Z_2, Z_3, and Z_4, are obtained from Figs. 3, 4, 5, and 6, respectively. Therefore,

P_o (from Equation (25) and Fig. 3) $= 31\,250 \times 0.1132 = 3\,535$ lb;

P_1 (from Equation (26) and Fig. 4) $= 31\,250 \times 0.1277 = 3\,990$ lb.;

M_o (from Equation (27) and Fig. 5) $= 806\,300 \times 0.00833 = 6\,715$ ft-lb.;

M_1 (from Equation (28) and Fig. 6) $= 806\,300 \times 0.0229 = 18\,470$ ft-lb.; and,

S_1 (from Equation (29) and Fig. 7) $= 31\,250 \times 0.1714 = 5\,355$ lb.

The moments, thrusts, and shears due to the weight of the arch are determined in a similar manner: $w_c = 150$ lb.; $w'_c = 150 \cos 47° = 102.5$ lb.; $w'_c\, tr = 6\,650$ lb.; and $w'_c\, tr^2 = 171\,700$ lb.

For $\frac{t}{r} = 0.10$ and $2\,\phi_1 = 143.5°$, the values of Z', Z'_1, Z'_2, Z'_3, and Z'_4 are obtained from Figs. 10(*a*), 10(*b*), 10(*c*), 10(*d*), and 10(*e*), respectively; thus,

P_o (from Equation (51) and Fig. 10(*a*)) $= 6\,650 \times 0.7568 = 5\,040$ lb.;

P_1 (from Equation (52) and Fig. 10(*b*)) $= 6\,650 \times 1.4263 = 9\,475$ lb.;

M_o (from Equation (53) and Fig. 10 (*c*)) $= -171\,700 \times 0.0208 = -3\,560$ ft-lb.;

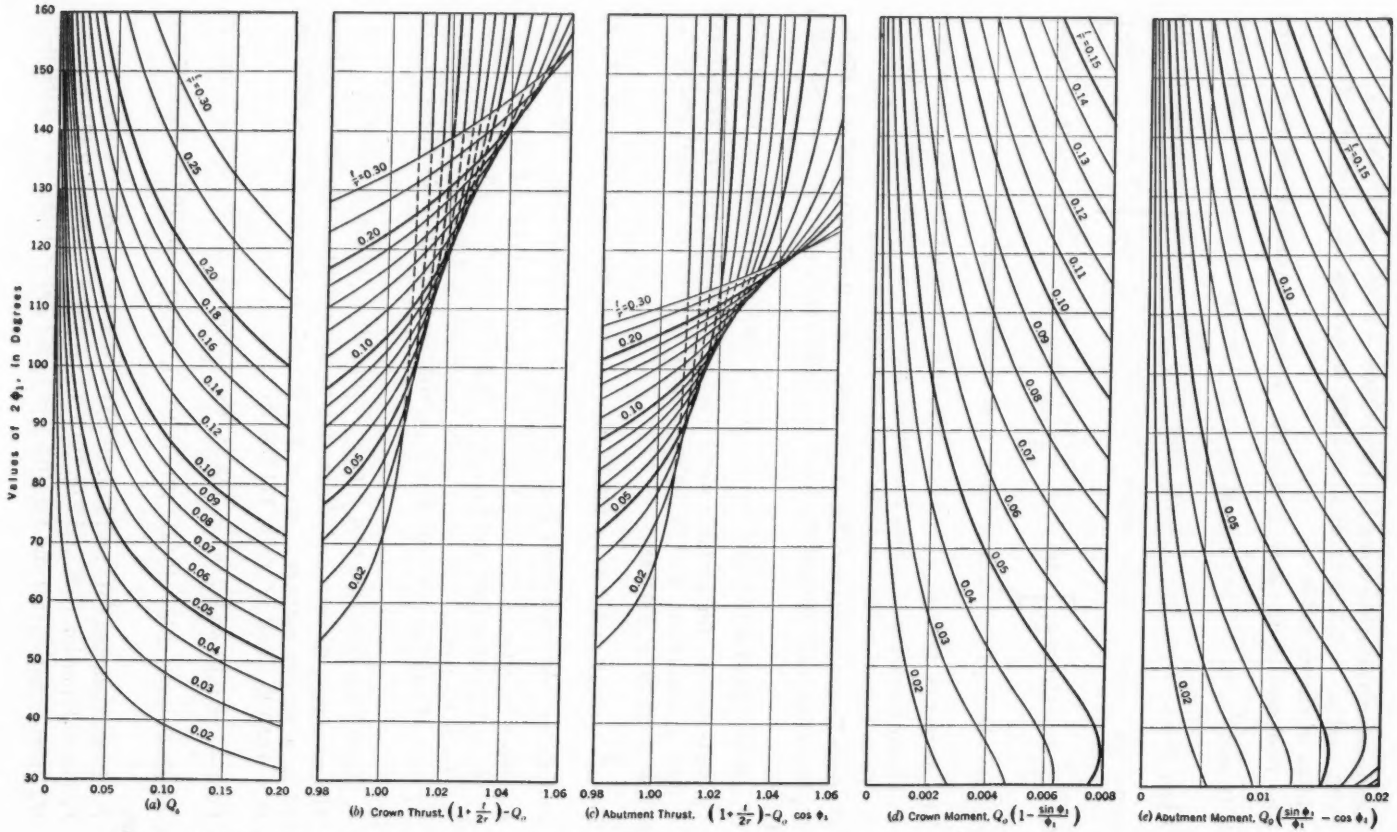
Values of $2\phi_1$, in Degrees
(a) Q_o
(b) Crown Thrust, $\left(1+\frac{t}{2r}\right)-Q_o$
(c) Abutment Thrust, $\left(1+\frac{t}{2r}\right)-Q_o \cos\phi_1$
(d) Crown Moment, $Q_o\left(1-\frac{\sin\phi_1}{\phi_1}\right)$
(e) Abutment Moment, $Q_o\left(\frac{\sin\phi_1}{\phi_1}-\cos\phi_1\right)$

FIG. 9.—CROWN AND ABUTMENT THRUSTS AND MOMENTS IN ARCH DAMS FOR WHICH $\frac{t}{r}$ DOES NOT EXCEED 0.3.

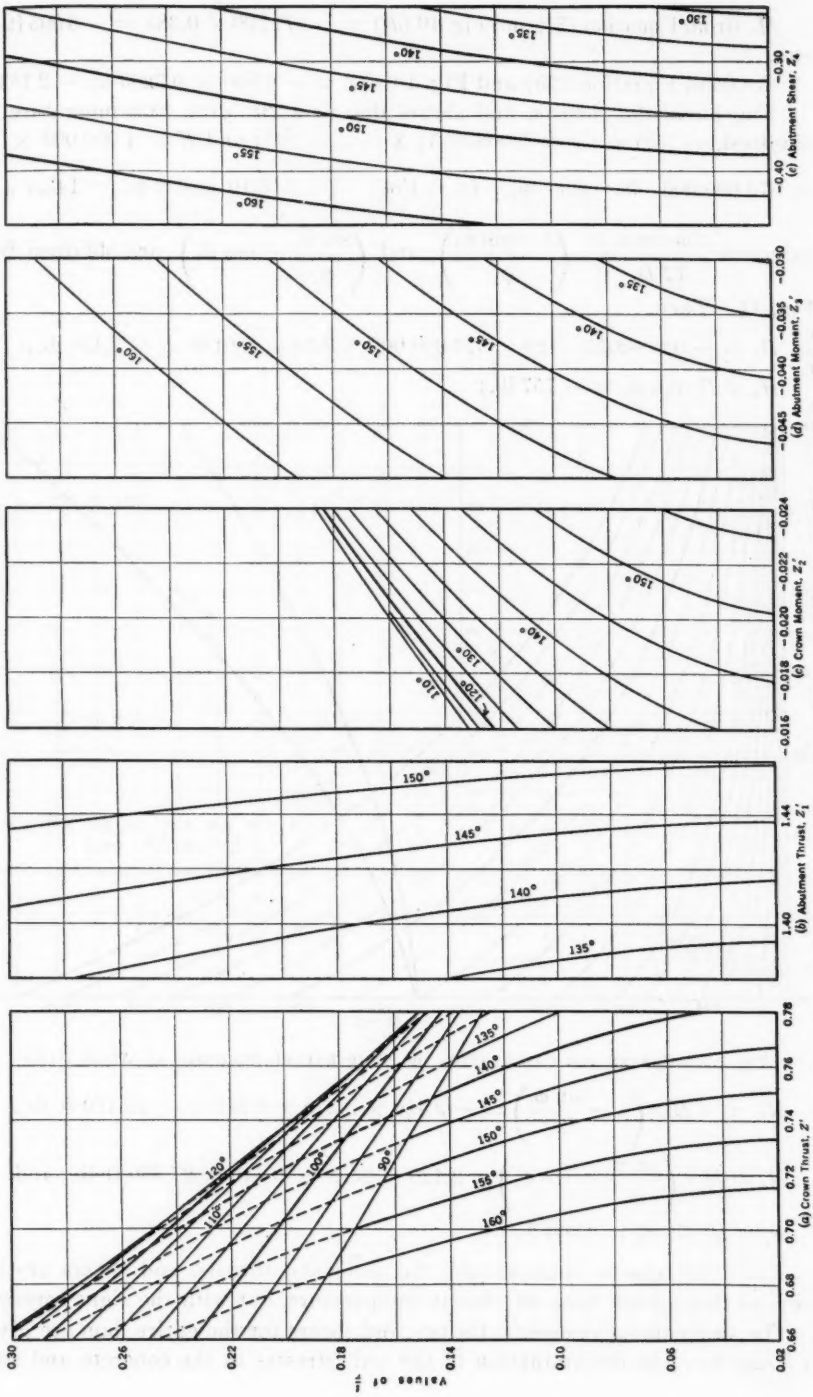

FIG. 10.—CROWN AND ABUTMENT THRUSTS AND MOMENTS DUE TO DEAD LOAD.

M_1 (from Equation (54) and Fig. 10 (*d*)) $= -171\,700 \times 0.385 = -6\,605$ ft-lb.; and,

S_1 (from Equation (55) and Fig. 10(*e*)) $= -6\,650 \times 0.3283 = -2\,183$ lb.

The moments, thrusts, and shears due to a 20° drop in temperature are obtained, as follows: $\varepsilon = 0.0000055$; $\Delta t = -20°$; and $E = 4\,000\,000 \times 144 = 576\,000\,000$ lb. per sq. ft. For $\frac{t}{r} = 0.10$ and $2\phi_1 = 143.5°$, the values of $\frac{2\phi_1 \sin\phi_1 t^2}{12 D_s r^2}$, $\left(\frac{1-\sin\phi_1}{\phi_1}\right)$, and $\left(\frac{\sin\phi_1}{\phi_1} - \cos\phi_1\right)$, are obtained from Fig. 11. Then,

$H_o = -0.0000055 \times 20 \times 576\,000\,000 \times 2.58 \times 0.0148 = -2\,420$ lb.;

$H_1 = H_o \cos\phi_1 = -757$ lb.;

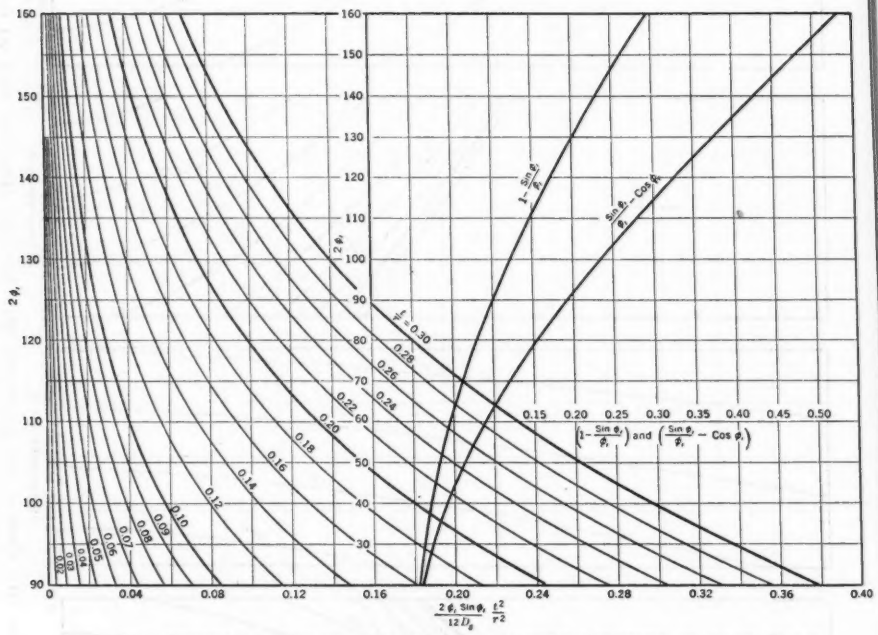

FIG. 11.—CURVES FOR CALCULATION OF TEMPERATURE STRESSES IN ARCH DAMS

$M_o = -H_o r\left(1 - \frac{\sin\phi_1}{\phi_1}\right) = -2\,420 \times 25.79 \times 0.242 = -15\,120$ ft-lb.;

$M_1 = H_o r\left(\frac{\sin\phi_1}{\phi_1} - \cos\phi_1\right) = 2\,420 \times 25.79 \times 0.446 = 27\,860$ ft-lb.; and,

$S_1 = H_o \sin\phi_1 = 2\,300$ lb.

For a 20° rise in temperature, the moments, thrusts, and shears are the same as those given for a 20° fall in temperature, but with the signs reversed.

The summary of moments, thrusts, and shears for the entire dam are given in Table 1. The determination of the unit stresses in the concrete and steel

TABLE 1.—SUMMARY OF MOMENTS, THRUSTS, AND SHEARS, DAM NO. 1*

Elevation	Loading	r_e, in feet	t, in feet	$\frac{t}{r}$	$2\phi_1$, in degrees	P_0, in pounds	P_1, in pounds	M_0, in foot-pounds	M_1, in foot-pounds	S, in pounds
180	Uniform water	26.50	2.00	0.079	143.0	25 730	25 900	−1 402	2 613	228
180	Variable water	26.50	2.00	0.079	143.0	3 365	3 790	6 310	17 160	5 070
180	Dead load	26.50	2 00	0.079	143.0	4 000	7 465	−2 545	−5 400	−1 732
180	20° temperature drop	26.50	2.00	0.079	143.0	−1 165	−370	−7 120	13 170	1 100
160	Uniform water	26.50	2.00	0.079	143.0	58 600	59 000	−3 192	5 940	511
160	Variable water	26.50	2.00	0.079	143.0	3 365	3 790	6 310	17 160	5 070
160	Dead load	26.50	2.00	0.079	143.0	4 000	7 465	−2 545	−5 400	−1 732
160	20° temperature drop	26.50	2.00	0.079	143.0	−1 165	−370	−7 120	13 170	1 100
140	Uniform water	27.08	2.58	0.100	143.5	92 300	93 200	−8 185	14 940	1 270
140	Variable water	27.08	2.58	0.100	143.5	3 535	3 990	6 715	18 470	5 355
140	Dead load	27.08	2.58	0.100	143.5	5 040	9 475	−3 560	−6 605	−2 183
140	20° temperature drop	27.08	2.58	0.100	143.5	−2 420	−757	−15 120	27 860	2 300
120	Uniform water	27.67	3.17	0.122	144.0	127 100	129 200	−16 790	31 300	2 442
120	Variable water	27.67	3.17	0.122	144.0	3 720	4 200	7 150	19 900	5 660
120	Dead load	27.67	3.17	0.122	144.0	6 183	11 800	−4 900	−7 840	−2 690
180	20° temperature drop	27.67	3.17	0.122	144.0	−4 260	−1 315	−27 040	48 900	4 040
100	Uniform water	28.25	3.75	0.142	144.5	162 300	165 500	−29 200	53 400	4 150
100	Variable water	28.25	3.75	0.142	144.5	3 860	4 380	7 540	21 200	5 920
100	Dead load	28.25	3.75	0.142	144.5	7 320	14 130	−6 450	−8 850	−3 182
100	20° temperature drop	28.25	3.75	0.142	144.5	−6 765	−2 060	−43 700	80 400	6 435
80	Uniform water	28.83	4.33	0.163	145.0	199 000	204 000	−46 070	84 250	6 725
80	Variable water	28.83	4.33	0.163	145.0	4 065	4 670	7 990	22 750	6 260
80	Dead load	28.83	4.33	0.163	145.0	8 435	16 470	−8 325	−9 335	−3 660
80	20° temperature drop	28.83	4.33	0.163	145.0	−9 910	−2 980	−65 200	119 700	9 450
60	Uniform water	29.42	4.92	0.183	145.5	236 000	243 000	−68 100	125 300	9 905
60	Variable water	29.42	4.92	0.183	145.5	4 250	4 840	8 480	24 450	6 590
60	Dead load	29.42	4.92	0.183	145.5	9 550	18 900	−10 510	−9 683	−4 140
60	20° temperature drop	29.42	4.92	0.183	145.5	−13 800	−4 090	−93 350	170 000	13 180
40	Uniform water	30.00	5.50	0.202	146.0	274 000	284 000	−96 300	176 900	13 130
40	Variable water	30.00	5.50	0.202	146.0	4 440	5 065	8 950	26 150	6 930
40	Dead load	30.00	5.50	0.202	146.0	10 630	21 350	−12 970	−9 730	−4 607
40	20° temperature drop	30.00	5.50	0.202	146.0	−18 500	−5 410	−125 700	231 000	17 700

* The signs of all moments and thrusts shown in this table are considered as positive unless marked with a minus sign.

is not considered in this paper as useful diagrams and tables for the determination of these combined bending and direct stresses are given in many textbooks and handbooks.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ANALYSIS OF CONTINUOUS FRAMES BY DISTRIBUTING FIXED-END MOMENTS

Discussion

BY MESSRS. A. W. EARL, A. FLORIS, I. M. NELIDOV, E. A. MACLEAN, GEORGE M. DILLINGHAM, AND DONALD E. LARSON

A. W. EARL,[33] M. AM. SOC. C. E. (by letter).[33a]—Structures composed of members the cross-section of which varies from point to point along their length are not, fortunately, of very common occurrence, since an accurate analysis entails considerable work. When such structures do occur, the writer believes that the method so clearly and concisely stated by Professor Cross, can be used to advantage and he, therefore, submits a development of the expressions for fixed-end moments, stiffness factors, and carry-over factors for members of variable cross-section, hoping they may prove useful.

The following notation will be used:

M_{AB} = moment at left end, A, of Member AB.
M_{BA} = moment at right end, B, of Member AB.
L = length of Member AB.
θ_A = change in slope of the tangent to the elastic curve of the member at A.
θ_B = change in slope of the tangent to the elastic curve of the member at B.
d = deflection of one end of a member with respect to the other end, measured perpendicularly to the original position of the member.

NOTE.—The paper by Hardy Cross, M. Am. Soc. C. E., was published in May, 1930, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: September, 1930, by Messrs. C. P. Vetter, L. E. Grinter, S. S. Gorman, A. A. Eremin and E. F. Bruhn; October, 1930, by Messrs. A. H. Finlay, R. F. Lyman, Jr., R. A. Caughey, Orrin H. Pilkey, and I. Oesterblom; November, 1930, by Messrs. Edward J. Bednarski, S. N. Mitra, Robert A. Black, and H. E. Wessman; January, 1931, by Messrs. Jens Egede Nielsen, F. E. Richart, and William A. Oliver; February, 1931, by Messrs. R. R. Martel and Clyde T. Morris; March, 1931, by Francis P. Witmer, M. Am. Soc. C. E.; May, 1931, by Messrs. T. F. Hickerson, F. H. Constant, W. N. Downey and E. C. Hartmann; September, 1931, by Messrs. Thomas C. Shedd, David M. Wilson, and Marshall G. Findley; November, 1931, by Messrs. George E. Large, and Sophus Thompson and R. W. Cutler; and January, 1932, by Alfred Gordon, Assoc. M. Am. Soc. C. E.

[33] Chf. Engr., The Sydney E. Junkins Co., B. C., Ltd., Vancouver, B. C., Canada.

[33a] Received by the Secretary December 8, 1931.

F_A = area of $\frac{M}{I}$ - diagram, due to the end moment, M_{AB}.

F_B = area of $\frac{M}{I}$ - diagram, due to the end moment, M_{BA}.

$m = \frac{F_A}{M_{AB}\,L}$ = area of $\frac{M}{I}$ - diagram for $M = 1$ and $L = 1$.

$n = \frac{F_B}{M_{BA}\,L}$ = area of $\frac{M}{I}$ - diagram for $M_{BA} = 1$ and $L = 1$.

s = stiffness factor.

u = distance of centroid of Area m, from the left end.

v = distance of centroid of Area n, from the right end.

F = area of $\frac{M}{I}$ - diagram for the applied loading.

$\bar{x}$ = distance of centroid of Area F from the right end.

The two basic propositions of the slope-deflection method may be expressed as follows:[34]

Area of $\frac{M}{EI}$ - diagram between B and A,

$$\theta_B - \theta_A = \int_B^A \frac{M dx}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots (96)$$

and moment of $\frac{M}{EI}$ - diagram between B and A about B,

$$d - L\,\theta_A = \int_B^A \frac{Mx\,dx}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots (97)$$

These expressions may be written,

$$\theta_B - \theta_A = \frac{1}{E}\left[- M_{AB}\,m\,L + M_{BA}\,n\,L - F \right] \ldots\ldots\ldots (98)$$

$$d - L\,\theta_A = \frac{1}{E}\left[- M_{AB}\,m\,(1-u)\,L^2 + M_{BA}\,n\,v\,L^2 - F\bar{x} \right] \ldots . (99)$$

Solving simultaneously for M_{AB} and M_{BA} and replacing $m\ u$ with $n\ v$, or *vice versa*, as required:

$$M_{AB} = \frac{E}{mL\,(1-u-v)}\left[\theta_A\,(1-v) + \theta_B\,v - \frac{d}{L}\right] - F\,\frac{\bar{x} - Lv}{mL^2\,(1-u-v)} \ldots (100)$$

and,

$$M_{BA} = \frac{E}{nL\,(1-u-v)}\left[\theta_A u + \theta_B\,(1-u) - \frac{d}{L}\right] + F\,\frac{L\,(1-u) - \bar{x}}{nL^2\,(1-u-v)} \ldots (101)$$

Equations (100) and (101) are the equations of slope deflection for a member of varying cross-section.

[34] For proof of the underlying propositions and development of the slope-deflection equations see "Analysis of Statically Indeterminate Structures by the Slope Deflection Method," by W. M. Wilson, F. E. Rickart, and Camillo Weiss, *Bulletin 108*, Eng. Experiment Station, Univ. of Illinois. The argument and sign convention of this *Bulletin* have been followed.

That $mu = nv$ may be proved, as follows:

$$mu = \frac{F_A\, u1}{M_{AB}\, L} = \frac{\int_0^L \frac{M}{I} \times dx}{M_{AB}\, L} \qquad nu = \frac{F_B\, vL}{M_{BA}\, L} = \frac{\int_0^L \frac{M}{I}\,(L-x)\,dx}{M_{BA}\, L}$$

$$M = \frac{M_{AB}}{L}\,(L - x) \qquad M = \frac{M_{BA}\, x}{L}$$

then, then,

$$mu = \frac{M_{AB}}{L} \frac{\int_0^L \frac{(L-x)\,x\,dx}{I}}{M_{AB}\, L} \qquad nv = \frac{M_{BA}}{L} \frac{\int_0^L \frac{(L-x)\,x\,dx}{I}}{M_{BA}\, L}$$

$$= \frac{1}{L^2} \frac{\int_0^L (L-x)\,x\,dx}{I} \qquad = \frac{1}{L^2} \frac{\int_0^L (L-x)\,x\,dx}{I}$$

therefore, $mu = nv$

Since, in the case of a member fixed at both ends, resting on unyielding supports, θ_A, θ_B, and $\frac{d}{L}$, each equal zero, the fixing moments at A and B, respectively, may be written equal to the last terms of Equations (100) and (101) as shown by Equations (102) (Table 18).

TABLE 18.—SUMMARY OF FACTORS FOR MOMENT DISTRIBUTION

Item.	End of beam	Fixing moment, M_F. (102)	Stiffness factor, S (103)	Carry-over factor, γ (104)
A.....	Left	$F\frac{\bar{x} - Lv}{mL^2\,(1-u-v)}$	$\frac{E\,(1-v)}{mL\,(1-u-v)}$	$\frac{v}{1-v}$
B.....	Right	$F\frac{L(1-u) - \bar{x}}{nL^2\,(1-u-v)}$	$\frac{E\,(1-u)}{nL\,(1-u-v)}$	$\frac{u}{1-u}$

The signs as given are correct for downward loading. In general, the sign of the fixing moment at the end of a member is opposite to that of the moment of the applied load about that end, clockwise moments being considered positive.

The stiffness factor is defined as the moment at one end of a member (resting on unyielding supports at both ends), necessary to produce unit rotation at that end when the other end is fixed. Then, the stiffness factor for the end, A, may be found by placing $\theta_A = 1$, $\theta_B = 0$, $F = 0$, and $\frac{d}{L} = 0$ in Equation (100) and the stiffness factor at the end, B, may be found by placing $\theta_B = 1$, $\theta_A = 0$, $F = 0$, and $\frac{d}{L} = 0$, in Equation (101). The results are as shown by Equations (103) (Table 18).

If E is constant, it may be made equal to unity in these equations, since comparative stiffness is all that is required.

The carry-over factor is defined as the ratio of the moment at the fixed end to the moment producing rotation at the rotating end of a member (which is on unyielding supports at both ends), if one end of the member is rotated while the other end remains fixed. Then, the carry-over factor at End A equals the ratio of M_{AB} to M_{BA} when θ_A, $\frac{d}{L}$, and F, are made equal to zero in Equations (100) and (101). Similarly, the carry-over factor at End B is the ratio of M_{BA} to M_{AB} when θ_B, $\frac{d}{L}$, and F, are made equal to zero in the same equations. Thus, they take the form of Equations (104) (Table 18.)

The quantities, m, n, v, u, F, and $\bar{x}$ having been determined, the fixing moments, stiffness, and carry-over factors are easily calculated and the analysis may proceed as for a structure with members of uniform cross-section. The "bulk" of the labor is in determining the six aforementioned quantities for each member of the structure. A well-ordered tabulation, Simpson's rule for approximate integration, and a calculating machine will be found very helpful.

The valuable tables [35] given by Walter Ruppel, Assoc. M. Am. Soc. C. E., may be used and the work thus greatly shortened. These tables give coefficients, s, t, u, v, p, and q, for use in the solution of continuous beams of variable cross-section by Fidler's method. Simple relationships exist which make them easily applicable to the slope-deflection method, or to the method of distributed moments. The tables cover a large variety of shapes and loadings.

As defined by Mr. Ruppel (using the writer's symbols) the heights, respectively, of the left and right characteristic points when L and W are equal to 1, are:

$$s = \frac{F\,\bar{x}}{L^3\,W m} \qquad \text{.....(105)}$$

and,

$$t = \frac{F\,(L - \bar{x})}{L^3\,W n} \qquad \text{.....(106)}$$

Solving simultaneously:

$$F = W\,L^2\,(s\,m + t\,n) \qquad \text{.....(107)}$$

and,

$$\bar{x} = \frac{s\,m\,L}{s\,m + t\,n} \qquad \text{.....(108)}$$

Thus, in terms of the coefficients given in Mr. Ruppel's table, m and n are obtained by dividing p and q, respectively, by the minimum moment of inertia of the member; u and v are obtained directly from the tables and F and $\bar{x}$ by substituting values for s and t in Equations (107) and (108). The tables are strictly accurate for beams of rectangular cross-section, but they may be used with satisfactory accuracy for T-beams.

[35] "Moments in Restrained and Continuous Beams," *Transactions*, Am. Soc. C. E., Vol. 90 (1927), 155 *et seq.*

A. FLORIS,[36] ESQ. (by letter).[36a]—The method of analyzing statically indeterminate structures by the aid of successive approximations, as proposed by the author, is not quite new.

The most systematic study, relative to the analysis of structures by means of statically indeterminate deformations instead of statically indeterminate forces, was made by the late Professor A. Ostenfeld, including a method of computation by steps for fixed as well as for movable joints.[37] This last method has been mentioned by one of the discussers.[37a] The writer would like to add two more methods due to well-known writers.

The method of Professor Akira Miura may be summarized as follows:[38] First, the joints are assumed to be restrained from rotating and the bending moments of the members are determined for this condition. In reality, the joints are not fixed, and it is necessary, therefore, to calculate, for each joint, the moments that resist rotation. These moments are termed "the moments of restraint." Now, if it is assumed that the moments of restraint are acting at the joints in the opposite direction, the corresponding rotations, and, consequently, the bending moments, of the members can be determined. By adding the values found in the first step to those obtained in the second step the final values for the actual conditions in the structure can be calculated. This analysis permits various simplifications in its detailed development and is, according to its author, more simple and practical than the method devised by Professor Ostenfeld.

The reviewer of an American textbook on structural analysis in which Professor Cross' method is given, states that: "The procedure is somewhat similar to that due to Dr. F. Hartmann [Vienna, Austria], in which the joints are locked by eliminating the end slopes of the members."[39] Since reference is not made to the original source the writer could not verify this statement.

The writer wishes merely to put these comments on record without going into detail as to the relative merits of the methods under consideration.

I. M. NELIDOV,[40] ASSOC. M. AM. SOC. C. E. (by letter).[40a]—Rigid frames can be classified as structures consisting of a number of members, which is usually less than that required by the theorem of Cauchy,[41] that is, the number of members equals three times the number of joints minus six. As a consequence, equilibrium under the action of a given set of forces is accomplished only after considerable deformation takes place. In this respect, the

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[36] Civ. Engr., Los Angeles, Calif.

[36a] Received by the Secretary December 19, 1931.

[37] "Die Deformationsmethode", von A. Ostenfeld, Berlin, 1926, pp. 17, 27, and 37.

[37a] *Proceedings*, Am. Soc. C. E., May, 1930, Papers and Discussions, p. 2056.

[38] "Theory of Rigid Frames" by Akira Miura, Tokyo, 1929 (in Japanese). Also, "Die neuzeitliche Entwicklung der Rahmentheorie in Japan", von Akira Miura, *Proceedings*, World Eng. Congress, Tokyo, 1929, Vol. VII, Pt. I, p. 422.

[39] *Concrete and Constructional Engineering* (London), August, 1931, Vol. XXVI, No. 8, p. 468.

[40] Senior Engr., Hydr. Structural Design, State Dept. of Public Works, Sacramento, Calif.

[40a] Received by the Secretary December 28, 1931.

[41] "Redundant Forces in Structural Mechanics," by the late V. L. Kirpitcheff, Prof. Emeritus at Polytechnic Institute at Kiev, 1903 Edition, p. 10.

point of view sometimes expressed that side-sway of the frame is of no importance apparently is in error.

Rigid frames generally fall into two distinct classes: First, those in which a given deflection of the joint at one end of the frame produces a known deflection at the remainder of the joints; and, second, those in which a given deflection of the joint at one end produces unknown deflections at the remainder of the joints.

Frames of the first class are: Two members connected at a joint and simply supported at opposite ends; three members meeting at a joint, or portals with parallel and non-parallel legs, saw-roof frames, and closed-ring frames; four members meeting at a joint, or closed rings, such as culverts, bins, etc.; and, finally, more than four members, such as structural building frames.

Portals consisting of four or more members with parallel or non-parallel legs, closed rings, and polygons, belong in the second class. Arches are included in the polygon classification. It should be noted that any frame may have rigid connections, but strictly rigid frames proper must have them in order not to become moving mechanisms.

All the frames of the first class can be solved with the aid of the method proposed by Professor Cross, by deflecting the structure a unit of distance, finding the resulting moments and shears, multiplying them by a certain coefficient, and equating them to acting forces.

With frames of the second class this procedure cannot be followed exactly because the deformations of the ring are known only when the total number of members is not greater than three. When this number is exceeded, the intermediate members "give" if the end members are undergoing a certain deformation.

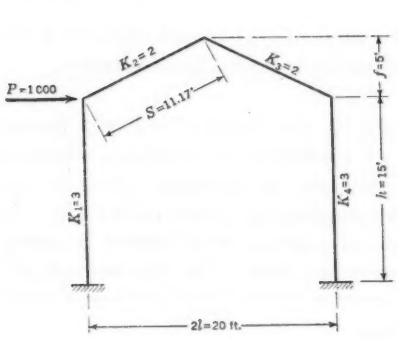

FIG. 55

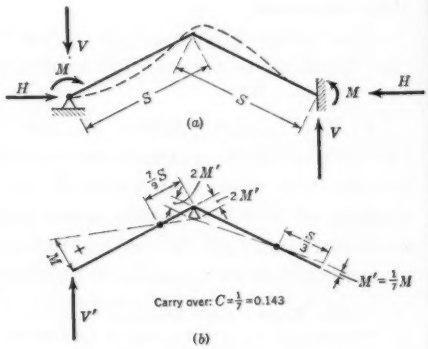

FIG. 56

To demonstrate the application of the Cross method to the second class of frames, with the number of members greater than three, the writer selected as the simplest case a symmetrical portal frame, consisting of four members and subjected to the action of a horizontal force, $P = 1$ kip, as shown on Fig. 55. Clockwise moments acting on the ends of a member are considered positive; moments in beams to the left of a section, or those which are

plotted in a moment diagram, are assumed positive when the curvature of the elastic line is convex downward.

The deflection of the frame under the load can be resolved into two component deflections: First, two equal deflections to the right, measured at the top of the columns; and, second, two equal but opposite deflections, one at the left column toward the right and one at the right column toward the left.

The top members of the portal above the columns can be replaced with one straight imaginary member connecting the top of the columns. This member should have properties equal to the replaced top of the portal, and its use will simplify the procedure. The properties to be found are: The new carry-over ratio; the new rigidity; and the forces and moments resulting from the deformation of the imaginary member.

To determine the carry-over ratio imagine the top portion of the frame hinged at the left end and fixed at the right, as shown on Fig. 56(*a*). From the condition which produces no displacement at the hinge, M' is determined with the aid of the conjugate-beam method: $\Delta L_v = 0$; therefore,

$$\frac{1}{EI}\left[\frac{1}{2}(M + 2M')\, s\, \frac{l}{3} - 2 \times 2M'\, sl + \frac{1}{2}(2M' + M')\, s\, \frac{5}{3} l\right] = 0 \quad \ldots(109)$$

Wherefrom: $M' = \frac{I}{7} M$, which gives the carry-over ratio, $c_i = -0.1429$, instead of $c = +0.5$ for a straight beam with constant moment of inertia. The moment diagram is shown on Fig. 56(*b*).

The deflection angle at the hinge is:

$$\Delta\phi = \frac{1}{EI}\left[\frac{1}{2}(M + 2M')\, s - 2 \times 2M's + \frac{1}{2}(2M' + M')\right]$$

$$= \frac{2}{7}\frac{Ms}{EI} = \frac{2}{7}\frac{M}{Ek} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(110)$$

in which, $k = \frac{I_2}{s_2} = \frac{I_3}{s_3} = 2$. This deflection angle must be equal to that of a straight imaginary beam hinged at the left end and fixed at the right, which is:

$$\Delta\phi_i = \frac{1}{4}\frac{M}{Ek_i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(111)$$

Since $\Delta\phi = \Delta\phi_i$:

$$\frac{2}{7}\frac{M}{Ek} = \frac{1}{4}\frac{M}{Ek_i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(112)$$

and $k_i = \frac{7}{8} k = \frac{7}{8} 2 = 1.75$.

After the carry-over ratio and rigidity of the imaginary member are determined, the procedure is as follows.

Step 1.—Deflect the frame toward the right an equal amount, Δ', at both columns, compute the fixed-end moments, release the joints, and balance the moments. Fixed-end moments for columns in this case are:

$$M = -6\, Ek_c\, \frac{\Delta'}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(113)$$

in which, $k_c = \frac{I_1}{l_1} = \frac{I_4}{l_4} = \frac{I}{h} = 3$. Assume an arbitrary value, $M = -10\,000$ ft-lb. Fixed-end moments for the imaginary member in this case are equal to zero. All computations for Step 1 are shown on Fig. 57(*a*).

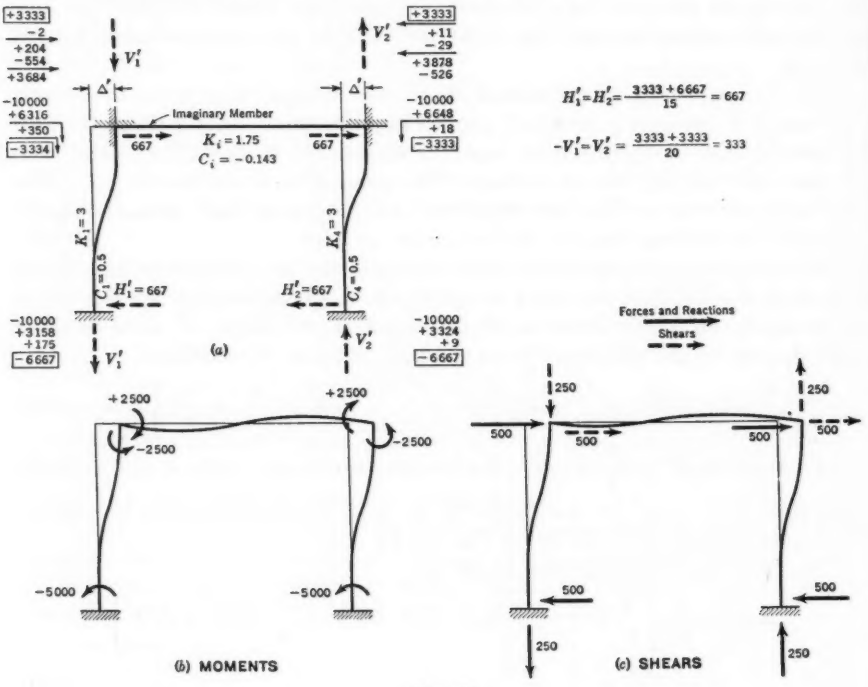

FIG. 57

Step 2.—Deflect the posts of the frame inward an equal amount, Δ''; compute the fixed-end moments; release the joints; and balance the moments. The fixed-end moments for columns in this case are expressed by:

$$M = \pm 6\,Ek_c\,\frac{\Delta''}{h} \qquad \text{.....(114)}$$

In this case a value is assumed arbitrarily as $M = \pm 5\,000$ ft-lb. Fixed-end moments in the imaginary member are computed in accordance with Fig. 58(*a*) and Fig. 58(*b*), as follows:

From the condition that $\Delta\phi = 0$, it follows that $M' = -M$; furthermore, $V = 0$ by condition of loading; and, since $M' = M - Hf$, it follows that $M = +\frac{Hf}{2}$. The horizontal shortening of the span, $2l$, is expressed by:

$$\Delta L_h = \frac{1}{EI}\left[2\,Ms\,\frac{1}{2}\,f - 2 \times \frac{1}{2}\,Hfs\,\frac{2}{3}\,f\right] = -\frac{Hf^2s}{6\,EI} = -\frac{Hf^2}{6\,Ek} \qquad \text{...(115)}$$

in which, $k = \frac{I}{s} = 2$.

On the other hand: $\frac{\Delta L_h}{2} = \Delta'' = -\frac{M_c h}{6 E k_c}$. As $\Delta'' = -\Delta L_h$:

$$\frac{Hf^2}{6Ek} = -\frac{2 M_c h}{6 E k_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (116)$$

or,

$$H = -\frac{2 M_c h k}{f k_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (117)$$

and,

$$M = \frac{Hf}{2} = -\frac{M_c h k}{f^2 k_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (118)$$

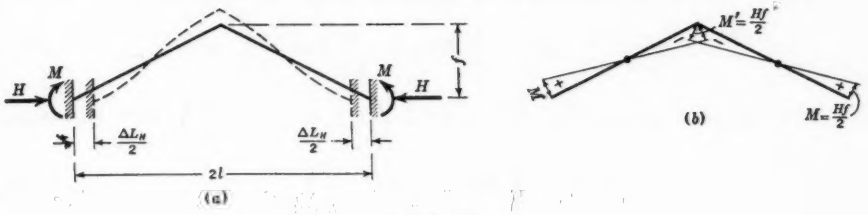

FIG. 58

For the left support, $M_c = -5\,000$ ft-lb.; $h = 15$ ft.; $k = 2$; $f = 5$ ft.; and $k = 3$. Therefore, the fixed-end moment in the imaginary member will be, $M = -\frac{-5\,000 \times 15 \times 2}{5 \times 3} = +10\,000$ ft-lb. The moments are balanced as in Fig. 59(*a*). The next step is to determine the coefficients, C_1 and C_2, by which the moments and shears just found are multiplied, in order to

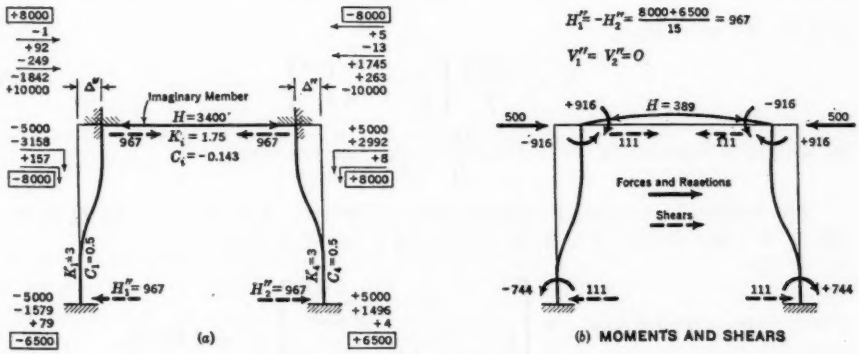

FIG. 59

obtain the true moments and shears. When the top of the left column is in equilibrium:

$$C_1 H_1' + (H_1'' + H) C_2 = P \quad \ldots\ldots\ldots\ldots\ldots\ldots (119)$$

and, when the top of the right column is in equilibrium:

$$C_1 H_2' - (H_2'' + H) C_2 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (120)$$

In the present case, $H_1' = H_2'$ and $H_1'' = -H_2''$; therefore, $C_1 = \frac{P}{2H_1'}$ and $C_2 = \frac{P}{2(H_1'' + H)}$.

Substituting the numerical values: $C_1 = \frac{1\,000}{2 \times 667} = 0.75$; and $C_2 = \frac{1\,000}{2(967 + H)} = \frac{500}{967 + H}$, Factor H is the comprehensive force in the imaginary member corresponding to the balanced moments at its ends, as found in Step 2.

From Fig. 58(a), the angle of rotation at the left support is:

$$\Delta\phi = \frac{1}{EI}\left(Ms - \frac{1}{2}Hfs\right) = \frac{1}{Ek}\left(M - \frac{1}{2}Hf\right) \quad \ldots\ldots\ldots (121)$$

For the adjacent column, $M'' = 2\,Ek_c\left(2\,\Delta\phi_c - 3\,\frac{\Delta''}{h}\right)$, and, therefore,

$$\Delta\phi_c = \left(M'' + 6\,Ek_c\,\frac{\Delta''}{h}\right)\frac{1}{4\,Ek} \quad \ldots\ldots\ldots\ldots (122)$$

Substituting $\frac{\Delta''}{h} = -\frac{M_c}{6\,Ek_c}$,

$$\Delta\phi_c = \frac{M'' - M_c}{4\,Ek_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots (123)$$

From the condition that $\Delta\phi = \Delta\phi_c$,

$$\frac{1}{Ek}\left(M - \frac{1}{2}Hf\right) = \frac{M'' - M_c}{4\,Ek_c} \quad \ldots\ldots\ldots\ldots (124)$$

and as $M'' = -M$,

$$H = \frac{2\,Mk}{f}\left[\frac{1}{k} - \frac{-1-u}{4\,k_c}\right] \quad \ldots\ldots\ldots\ldots\ldots (125)$$

in which, u represents the ratio, $\frac{M_c}{M}$, for convenience.

The vertical reaction, V, equals zero, as before. Let $M = +\,8\,000$ ft-lb.; $M_c = -\,5\,000$ ft-lb.; $k = 2$; $k_c = 3$; and $f = 5$ ft.; then,

$$H = 2 \times 8\,000 \times \tfrac{2}{5}\left[\tfrac{1}{2} - \frac{-1 - \frac{-5\,000}{8\,000}}{4 \times 3}\right] = +\,3\,400 \text{ lb.}$$

and,

$$C_2 = \frac{500}{967 + 3\,400} = 0.1145$$

The actual moments and shears are shown on Fig. 57(a) and Fig. 59(b). By adding these values, the resultant moments and shears are shown in

Fig. 60. With all the external moments and forces known, the moment at the top of the portal is found to be:

$$M_C = M_A - Vl + H_1(h+f) - Pf = -5\,744 - 250 \times 10 + 611\,(15+5) - 1\,000 \times 5 = -1\,024 \text{ ft-lb.}$$

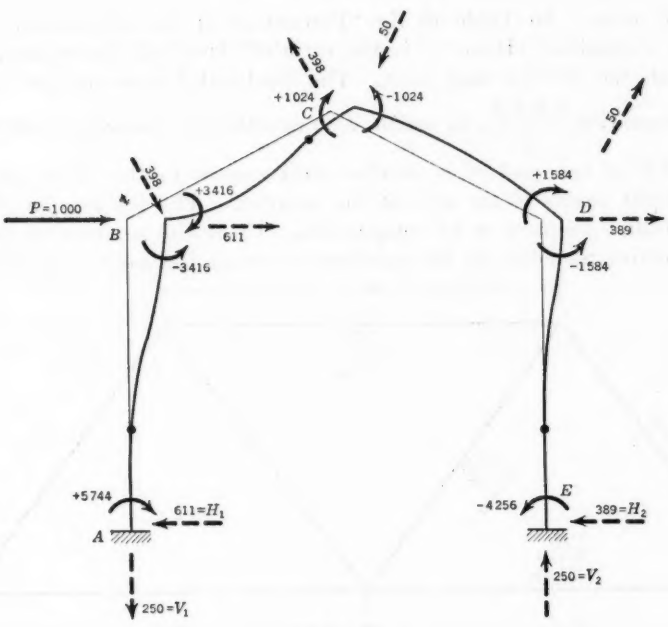

FIG. 60

An analytical check[42] gives the following results: $M_A = 5\,744$ ft-lb.; $M_B = 3\,431$ ft-lb.; $M_C = -1\,036$ ft-lb.; $M_D = -1\,590$ ft-lb; and $M_E = -4\,256$ ft-lb.

The foregoing computations show that Professor Cross' method can be applied to both classes of rigid frames.

E. A. MACLEAN,[43] ESQ. (by letter).[43a]—The following example of the Cross method of distributing moments, as applied to a truss, is submitted for determining secondary stresses. In spite of the simplicity of the Cross method, it becomes tedious when applied to a large truss, but the chances of error can be greatly reduced by systematizing the work. The method of laying out the computations, as shown in the example herein, will be found to be very helpful, not only in the computation of the secondary stresses in a truss, but in computing the moments in a multi-story bent or any other complicated structure.

[42] "Rahmenformeln," von A. Kleinlogel, Fifth Edition, p. 295.
[43] Designer, State Highway Comm., Augusta, Me.
[43a] Received by the Secretary January 4, 1932.

The problem is taken from a standard textbook[44] in which a solution of the secondary stresses is made by one of the older methods. The necessary data are shown in Fig. 61, in which, I = moments of inertia; l = length, in inches; $K = \frac{I}{l}$; c = distance from the neutral axis to the outer fiber; and s = unit stress. In Table 19 the "Percentage of the unbalanced moment going to a member" (Item 1), is the ratio of the K of the member to the total of all the K's for that joint. The fixed-ended moments are obtained from the equation, $\frac{6\,DEK}{l}$, in which, E = modulus of elasticity; and D = the deformation of one end of a member with respect to the other end measured at right angles to the axis of the member. The D's may be obtained from a Williot diagram or by computation. Clockwise rotation of the joint causes positive moments in the members entering the joint. In Table 19,

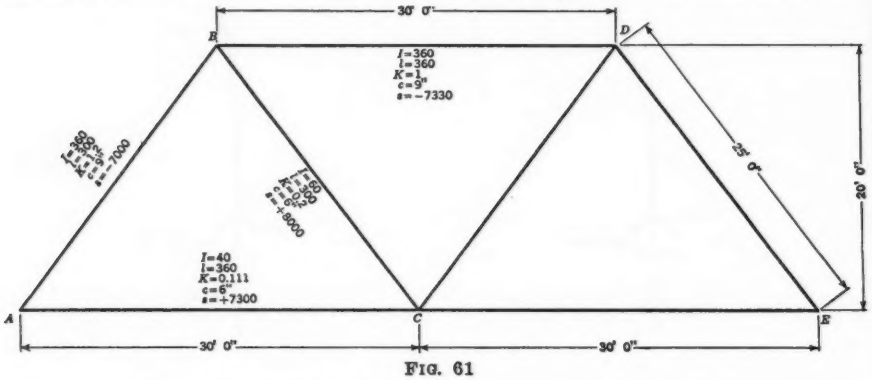

FIG. 61

the computations for Joints D and E are omitted because the moments are the same as for Joints B and A, respectively, with opposite signs.

The fixed-ended moments are entered in the table (Item 2), under the proper member and are added across algebraically for each joint, in order to obtain the unbalanced moment. This unbalanced moment is balanced at each joint (Item 3). Each member will get the percentage shown at the top of the table (Item 1). These "balancing" moments are then distributed on the following line, one-half the "balancing" moment at one end of a member being the "distributing" moment at the other end. In making up Table 19 it should be noted that a member (for example, the end-post, AB) is identified as AB under Joint A and as BA under Joint B. The "distributing" moments (Item 4) are added across to obtain a new unbalanced moment, and one cycle is finished. As the work proceeds, every box in the table is filled, except those with the dotted lines, or better still, diagonal lines. This system eliminates as far as possible the chances of making an error by omitting a part of a step, and it also enables the work to be checked easily. The final moment in the members is obtained by adding each column of moments algebraically (Item 13).

[44] "Steel Mill Buildings," by Milo S. Ketchum, M. Am. Soc. C. E., p. 309.

The following secondary stresses were obtained which are, of course, very close to the results shown in the original problem: $BA = -630$; $CA = +1780$; $CB = +826$; and $BD = -700$.

In teaching work the writer found that undergraduate as well as graduate students readily grasped the Cross method and obtained a much clearer con-

TABLE 19.—BALANCING MOMENTS BY CROSS METHOD

Item	Member	JOINT *A*			JOINT *B*				JOINT *C*				
		AB	*AC*	Unbalanced	*BA*	*BC*	*BD*	Unbalanced	*CA*	*CB*	*CD*	*CE*	Unbalanced
1	Percentage of unbalanced moment going to member	91.7	8.3	.....	50	8.3	41.7	.....	17.8	32.2	32.2	17.8	.....
2	Fixed moment.	−156.3	−17.7	−174.0	−156.3	−13.8	0.0	−170.1	−17.7	−13.8	+13.8	+17.7	0
3	Balancing....	+159.5	+14.5	.....	+85.0	+14.1	+71.0	.....	0	0	0	0	.....
4	Distributing..	+42.5	0	+42.5	+79.75	0	−35.5	+44.25	+7.25	+7.05	−7.05	−7.25	0
5	Balancing....	−39.0	−3.5	.....	−22.12	−3.68	−18.45	.....	0	0	0	0	.....
6	Distributing..	−11.06	0	−11.06	−19.5	0	+9.22	−10.28	−1.75	−1.84	+1.84	+1.75	.....
7	Balancing....	+10.15	+0.91	.....	+5.14	+0.86	+4.28	.....	0	0	0	0	.....
8	Distributing..	+2.57	0	+2.57	+5.06	0	−2.14	+2.92	+0.45	+0.43	−0.43	−0.45	0
9	Balancing....	−2.35	−0.22	.....	−1.46	−0.24	−1.22	.....	0	0	0	0	.....
10	Distributing..	−0.73	0	−0.73	−1.17	0	+0.61	−0.56	−0.11	−0.12	+0.12	+0.11	0
11	Balancing....	+0.67	+0.06	.....	+0.28	+0.05	+0.23	.....	0	0	0	0	.....
12	Distributing..	+0.14	0	+0.14	+0.34	0	−0.12	+0.22	+0.03	+0.025	−0.025	−0.03	.....
13	Totals, in kips.	+6.09	−5.95	.....	−24.98	−2.71	+27.91	.....	−11.83	−8.26	+8.26	+11.83	.....

ception of the action of undeterminate structures as they followed through this method, than is possible by the more or less blind application of formulas. Students are appreciative, of course, of the ease of its application, particularly after using some of the more formal methods. Once the Cross method is grasped, it can be called to mind after only a few minutes' thought without any reference to notes or complicated formulas.

GEORGE M. DILLINGHAM,[45] ASSOC. M. AM. SOC. C. E. (by letter).[45a]—For the analysis of continuous frames subjected to changes in loading, the method developed by Professor Cross is made readily applicable by the following steps:

(1) Determine the structural properties for a given frame; that is, dimensions; conditions of end restraint; the values of I, K, and the "carry-over" factors of the members, etc.

(2) Apply a "trial fixed-end moment" at the end of a member, either positive or negative, and of any convenient magnitude and unit.

[45] Chicago, Ill.

[45a] Received by the Secretary January 9, 1932.

(3) Distribute this "trial fixed-end moment" throughout the frame by the author's method, and tabulate the "trial final moment" thus induced for each end of all members.

(4) In like manner apply, in turn, "trial fixed-end moments" at the other end of the member mentioned in Step (2), and at each end of all other members; and tabulate for each application the corresponding "trial final moments."

(5) Determine all the "actual fixed-end moments" induced by following Step (3) for a specific loading.

(6) Determine the "actual final moments" for the given loading by the use of the following proportion:

$$\frac{\text{Actual final moment}}{\text{Actual fixed-end moment}} = \frac{\text{Trial final moment}}{\text{Trial fixed-end moment}}$$

Tabulate the final moments caused by each of the several actual fixed-end moments; add these algebraically for the end of each member, and record the total final moment.

(7) Change the loading, and repeat Steps (5) and (6).

Example.—In demonstration of Steps (1) to (7) the structure shown in Fig. 62[46a] is analyzed herewith. The signs of moments are determined by

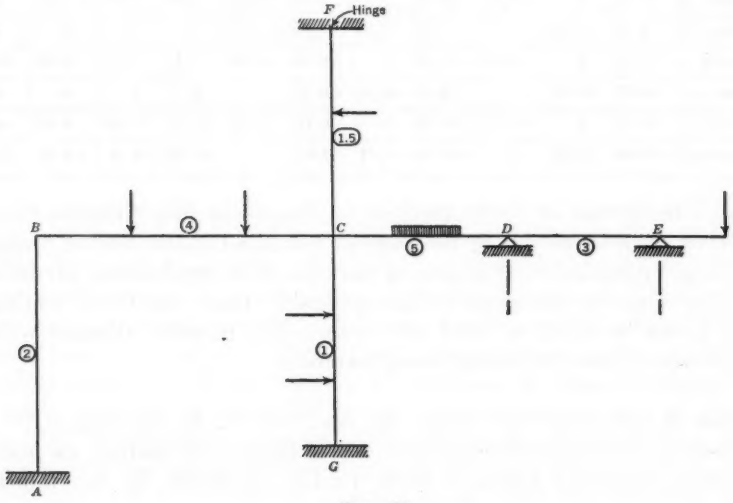

FIG. 62

the convention which designates, as positive, clockwise rotation at a joint due to externally applied forces.

Step (1): For greater ease in distributing the fixed-end moments, the K of the member, CF, is reduced from 2.0 to 1.5; the fixed-end moment at F is eliminated; and the fixed-end moment at C in CF is changed from −80

[46a] *Proceedings*, Am. Soc. C. E., May, 1930, Papers and Discussions, p. 922.

TABLE 20.—ARRANGEMENT OF COMPUTATIONS FOR EXAMPLE

Item	Joint	B		C			D		E		F	G	
	Member	A		CB		CF	DC	DE	ED		FC	GC	
	Distribution factor												
	Fixed-end moment												
	Trial fixed-end moment												
	Trial final moment												
	Actual fixed-end moment												
	Actual final moment												

TABLE 20.—ARRANGEMENT OF COMPUTATIONS FOR EXAMPLE

Item	A	B		C				D		E	F	G	Joint
1	AB	BA	BC	CB	CD	CF	CG	DC	DE	ED	FC	GC	Member
	FINAL MOMENTS INDUCED BY THE TRIAL FIXED-END MOMENT OF 1 000 FOOT-POUNDS												
2	2	2	4	4	5	1.5	1	5	3	3	1.5	1.0	K.
3	100.0%	33.33%	66.67%	34.77%	43.48%	13.05%	8.70%	62.5%	37.5%	0.0%	0.0%	100.0%	Distribution factor.
4	.....	1 000.0	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	Trial moment at *BA*.
5	+177.6	−644.6	+644.6	+221.9	−138.5	−50.1	−33.3	−26.1	+26.1	0.0	0.0	−16.6	
6	.....	.....	−1 000.0	.....	.....	.....	.....	.....	.....	.....	.....	.....	Trial moment at *BC*.
7	+177.6	+355.4	−355.4	+221.9	−138.5	−50.1	−33.3	−26.1	+26.1	0.0	0.0	−16.6	
8	.....	.....	.....	−1 000.0	.....	.....	.....	.....	.....	.....	.....	.....	Trial moment at *CB*.
9	−33.2	−66.8	+66.8	−665.7	+415.3	+150.3	+100.2	+77.8	−77.8	0.0	0.0	+50.2	
10	.....	.....	.....	.....	−1 000.0	.....	.....	.....	.....	.....	.....	.....	Trial moment at *CD*.
11	−33.2	−66.8	+66.8	+334.3	−584.7	+150.3	+100.2	+77.8	−77.8	0.0	0.0	+50.2	
12	.....	.....	.....	.....	.....	−1 000.0	.....	.....	.....	.....	.....	.....	Trial moment at *CF*.
13	−33.2	−66.8	+66.8	+334.3	+415.3	−849.7	+100.2	+77.8	−77.8	0.0	0.0	+50.2	
14	.....	.....	.....	.....	.....	.....	−1 000.0	.....	.....	.....	.....	.....	Trial moment at *CG*.
15	−33.2	−66.8	+66.8	+334.3	+415.3	+150.3	−899.8	+77.8	−77.8	0.0	0.0	+50.2	
16	.....	.....	.....	.....	.....	.....	.....	−1 000.0	.....	.....	.....	.....	Trial moment at *DC*.
17	+11.5	+22.9	−22.9	−115.3	+201.6	−51.7	−34.6	−337.3	+337.3	0.0	0.0	−17.2	
18	.....	.....	.....	.....	.....	.....	.....	.....	−1 000.0	.....	.....	.....	Trial moment at *DE*.
19	+11.5	+22.9	−22.9	−115.3	+201.6	−51.7	−34.6	+662.7	−662.7	0.0	0.0	−17.2	
	FINAL MOMENTS INDUCED BY THE ACTUAL FIXED-END MOMENTS												
20	−17.76	−35.54	+35.54	−22.19	+13.85	+5.01	+3.33	+2.61	−2.61	0.0	0.0	+1.66	*BC* + 100.
21	−3.32	−6.68	+6.68	−66.58	+41.53	+15.03	+10.02	+7.78	−7.78	0.0	0.0	+5.02	*CB* — 100.
22	+6.64	+13.36	−13.36	−66.85	+116.95	−30.06	−20.04	−15.56	+15.56	0.0	0.0	−10.04	*CD* + 200.
23	−3.65	−7.35	+7.35	+36.77	+45.68	−93.47	+11.02	+8.56	−8.56	0.0	0.0	+5.52	*CF* — 110.
24	−1.66	−3.34	+3.34	+16.72	+20.76	+7.51	−44.99	+3.89	−3.89	0.0	0.0	+2.51	*CG* — 50.
25	+1.15	+2.29	−2.29	−11.53	+20.16	−5.17	−3.46	−33.73	+33.73	0.0	0.0	−1.72	*DC* — 100.
26	+0.06	+0.11	−0.11	−0.58	+1.01	−0.26	−0.17	+3.31	−3.31	0.0	0.0	−0.09	*DE* — 5.
27	.....	.....	.....	.....	.....	.....	.....	.....	.....	−10.0	.....	.....	*ED* −10.
28	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	+50.00	*GC* + 50.
29	−18.54	−37.15	+37.15	−114.24	+259.94	−101.41	−44.29	−23.14	+23.14	−10.0	0.0	+52.86	Final moments.

to −110 (see Fig. 62). These changes are made possible by the absence of restraint at the hinge, *F*. In all other respects, the data given by the author are used unchanged.

Steps (2), (3), and (4): In Table 20, Items 1 to 19 show the final moment induced at each end of each member by the trial fixed-end moment of −1 000.0, which is applied successively at Joints *B*, *C*, and *D*, as indicated. In preparing Table 20, the trial moment was applied to *AB* at *B*, to *CB* at *C*, and to *CD* at *D*; and the distribution process was carried out in the usual manner for each of these applications. From the results of these three applications, the values of the final moments for the other locations of the trial fixed-end moment were written by inspection.

Step (5): See Step (1).

Step (6): In Table 20, Items 20 to 29 show the final moments induced at each end of each member by the actual fixed-end moments given in Fig. 62.

The similarity of the general procedure described in this example to influence line diagram construction is easily recognized. Having once performed Steps (2), (3), and (4), loading changes may be made as desired, and final moments for each loading determined quickly and with a minimum of effort.

The distribution of Step (3) may be performed either by the method given in the paper under discussion, or by the method (termed the "Exact Method") described by Professor Cross in his paper[46] entitled "Continuity as a Factor in Reinforced Concrete Design."

DONALD E. LARSON,[47] JUN. AM. SOC. C. E. (by letter).[47a]—In presenting the method of moment distribution, Professor Cross has barely hinted at the possibilities of the more general method of moment and force distribution which includes the distribution of unbalanced joint forces as well as moments. This general method must be used in analyzing structures in which the joints are subject to linear displacement as well as rotation. A continuous series of arches supported on long slender piers is a structure of this type. Loads applied to the arches produce both rotation and horizontal translation of the joints. For this reason, the processes of moment distribution and force distribution cannot be carried out independently of each other. When the moments at a joint are balanced, the horizontal forces at the ends of the members intersecting at that joint are changed and must be adjusted accordingly. In the same way, the moments at the ends of these members must be adjusted when the horizontal forces at the joint are balanced.

After considerable experimenting, the writer has worked out a fairly convenient procedure for analyzing multiple-span arches by distributing fixed-end horizontal forces and fixed-end moments. The method has several desirable features even if it is complicated to a certain extent by the adjustments previously mentioned. An outline of the procedure follows.

[46] *Proceedings*, Am. Concrete Inst., 1929, p. 689.

[47] Asst. Research Engr., Chicago Bridge & Iron Works, Chicago, Ill.

[47a] Received by the Secretary January 9, 1932.

Step 1.—

(*a*) Compute the fixed-end horizontal forces at the ends of each arch, due to loads on the arch.

(*b*) Compute the fixed-end moments at the ends of each arch, resulting from loads on the arch.

Step 2.—

(*a*) Distribute the unbalanced horizontal force at each joint among the connecting members in proportion to their resistance to horizontal deflection. The percentage of the unbalanced force distributed to each member is equal to $\frac{J}{\Sigma J}$, in which, J is the horizontal force required to deflect the near end of the member one unit when applied along the centroidal axis of the member, the far end of the member being fixed.

(*b*) Distribute the unbalanced moment at each joint among the connecting members in proportion to their resistance to rotation. The percentage distributed to each member is equal to $\frac{N}{\Sigma N}$, in which, N is the moment required to rotate the near end of the member one unit when the far end is fixed.

Step 3.—

(*a*) The horizontal force distributed to each member in Step 2 (*a*) is carried over to the opposite end of the member with opposite sign. This is to keep the member in static equilibrium.

(*b*) A certain proportion, r, of the moment distributed to each member in Step 2 (*b*) is carried over to the far end of the member. This "carry-over factor," r, is the same as the one defined by the author. It may be either positive or negative, depending upon the geometric properties of the member.

Step 4.—

(*a*) Compute the horizontal thrust at the end of each member produced by the moments distributed to that member in Step 2 (*b*). These thrusts are equal to the product of this moment by h, in which, h is the horizontal force produced by a unit moment applied to one end of the member.

(*b*) Compute the moments at the ends of each member produced by the horizontal force distributed to that member in Step 2 (*a*). These moments are equal to the product of the force by d, in which, d is the vertical distance from the joint to the elastic center of the member.

Step 5.—

(*a*) The horizontal forces obtained in Step 3 (*a*) and Step 4 (*a*) are added algebraically.

(*b*) The moments obtained in Step 3 (*b*) and Step 4 (*b*) are added in the same manner.

These steps form a complete cycle in the process of moment and force distribution. Step 5 (*a*) gives a new set of horizontal forces at each joint. The resulting unbalanced force is again distributed, as in Step 2 (*a*). Step 5 (*b*) gives a new set of moments at each joint and the unbalanced

moment is again distributed as in Step 2 (*b*). The foregoing procedure is then repeated as many times as required to produce results of the desired accuracy.

Formulas for Constants.—The constants defined in Steps 1 to 5, inclusive, may be computed by the relations given in Equations (126) to (134). The derivation of these relations is based upon the assumptions: First, that plane sections remain plane; and, second, that the material obeys Hooke's law. The subscript, *a*, is used to indicate that the constant applies to an arch, and the subscript, *p*, to indicate that the constant applies to a pier.

Constants for arches are expressed by the following equations:

$$d_a = \frac{\sum y \frac{ds}{I}}{\sum \frac{ds}{I}} \quad \ldots\ldots (126)$$

$$J_a = \frac{1}{\sum y^2 \frac{ds}{I} - d^2_a \sum \frac{ds}{I}} \quad \ldots\ldots (127)$$

$$N_a = \frac{1}{\sum \frac{ds}{I}} + \frac{L^2}{4 \sum x^2 \frac{ds}{I}} + d^2_a J_a \quad \ldots\ldots (128)$$

$$r_a = 1 - \frac{L^2}{2 N_a \sum x^2 \frac{ds}{I}} \quad \ldots\ldots (129)$$

and,

$$h_a = \frac{d_a J_a}{N_a} \quad \ldots\ldots (130)$$

Constants for piers are:

$$d_p = \frac{\sum y \frac{ds}{I}}{\sum \frac{ds}{I}} \quad \ldots\ldots (131)$$

$$J_p = \frac{1}{\sum y^2 \frac{ds}{I} - d^2_p \sum \frac{ds}{I}} \quad \ldots\ldots (132)$$

$$N_p = \frac{1}{\sum \frac{ds}{I}} + d^2_p J_p \quad \ldots\ldots (133)$$

and,

$$h_p = \frac{d_p J_p}{N_p} \quad \ldots\ldots (134)$$

The factor, r_p, corresponding to r_a in Equation (129) is not required when piers are fixed at the base. The geometric properties which are needed for computing the arch constants are shown in Fig. 63. The properties used in computing constants for the piers are shown in Fig. 64.

Signs.—The convention of signs adopted by the writer is similar to that introduced by the author; that is, positive moment in the arches corresponds to compression at the extrados; positive moment in the piers corresponds to

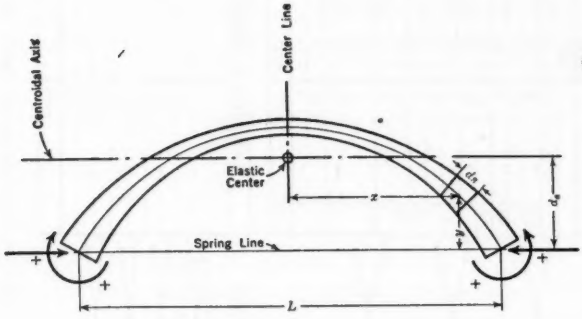

FIG. 63

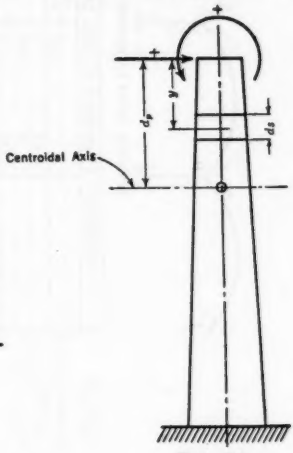

FIG. 64

compression on the left side of the pier; horizontal forces acting inward on the end of the arch are positive; and a force acting to the right on the top of the pier is positive. In balancing moments and horizontal forces at the joint, the pier is considered to be on the same side of the joint as the arch at the left. A positive moment applied to an arch or a pier produces a positive horizontal force; and a positive horizontal force applied to an arch or pier produces a positive moment. The directions of positive moments and forces are indicated by arrows in Fig. 63 and Fig. 64. The constants, except r, are always positive when used with the foregoing convention of signs.

Illustrative Example.—The analysis of a three-span arch by the method outlined, is illustrated in Fig. 65. For the sake of simplicity, a structure with arches all alike and piers of the same height has been chosen. The ends of the outer arches and the bases of the piers are fixed. The problem is to determine the moments and horizontal forces at the ends of the arches and piers.

Since the only purpose of the problem is to illustrate the method used in distributing the fixed-end moments and forces, the actual dimensions of the arches and piers are not given. The constants required for the analysis are: For arches, $d_a = 39.7$; $J_a = 0.00111$; $N_a = 2.53$; $r_a = +0.491$; and $h_a = 0.0175$; and for piers, $d_p = 47.5$; $J_p = 0.00643$; $N_p = 21.60$ and $h_p = 0.0141$. These constants represent a continuous arch series supported by piers which are relatively long and slender.

Step	DISTRIBUTION OF HORIZONTAL FORCES TO ENDS OF ARCHES						
1a				→ 65.5		65.5 ←	
2a				← 8.4	→ 8.4		
3a			← 8.4		8.4 →		
4a			0		0		
			← 2.4		2.4 →		
5a			0		0		
			→ 6.0		← 2.4		
2a		0.8 ←		2.4 →		← 2.4	
3a	→ 0.8		1.5 →		← 1.5		6.0 →
4a	0		→ 1.5	0.8 →		1.5 ←	
	→ 0.4	0.4 ←			← 2.7		
5a	→ 1.2		→ 0.8	2.7 →		2.7 ←	
2a	0	0.4 ←			← 2.7		
3a	→ 0.4		1.9 →		← 1.9		1.2 →
4a	0	0.4 ←		0.4 →		1.9 ←	
	→ 0.3		0.4 →		← 0.4		0 →
5a	→ 0.7	0.3 ←		0.3 →		0.4 ←	
2a	0	0.4 ←	→ 1.2		← 0.4		1.5 →
3a	→ 0.4		0.2 →	1.1 →		1.5 ←	
4a	0	0.3 ←			← 0.2		
	→ 0.3		0.6 →	0.4 →		0.2 ←	
5a	→ 0.7	0.3 ←			← 0.6		0.2 →
2a	0	0.2 ←	→ 0.1	0.5 →		0.6 ←	
Σ	→ 2.6	2.8 ←	→ 6.3	7.0 ←	→ 55.9	56.4 ←	

Add Figures in These Rows to Obtain the Horizontal Forces and Moments at Ends of Arches

h_2=0.0175
d_2=39.7
Multiply by h_2
Multiply by d_2

Step	DISTRIBUTION OF MOMENTS TO ENDS OF ARCHES							
					−1440		+1430	1b
				−137	+ 137			2b
			− 67		0		+ 67	3b
			0	+334	− 334		0	4b
			+334	0	0		− 334	
			+267		+334		− 267	5b
		Multiply by ⅓	− 25	×⅓	+ 156	×⅓	− 156	2b
			+ 77	0	− 12		0	3b
		+ 25	− 32		− 59		+ 59	4b
			− 59		− 32		0	
			− 34		0		+ 59	
			− 14		−103		− 18	5b
		+ 32	− 16		− 24		+ 24	2b
		+ 16	− 16		− 8		0	3b
		0	− 12		0		+ 12	
		+ 16	− 16		+ 75		− 75	4b
		0	+ 75		− 16		0	
		+ 16	+ 47		+ 51		− 75	
		+ 16	− 15		+ 37		− 37	5b
		0	+ 18		− 7		0	2b
		+ 16	− 16		− 8		+ 8	3b
		0	− 8		− 16		0	4b
		+ 16	− 6		− 31		+ 8	
		7	− 7		− 1		+ 1	5b
		+127	+231		+282		+1813	2b
								Σ
							+1072	

Step	DISTRIBUTION OF HORIZONTAL FORCES TO TOPS OF PIERS
1a	0
2a	
4a	48.7 ←
	16.5 ←
2a	8.7 →
4a	18.8 →
2a	10.8 ←
4a	2.9 →
2a	1.0 ←
4a	4.4 →
2a	2.9 ←
Σ	48.9 ←

Add All Figures to Obtain the Horizontal Forces and Moments at Tops of Piers

h_3=0.0141
d_3=47.5
Multiply by h_3
Multiply by d_3

Step	DISTRIBUTION OF MOMENTS TO TOPS OF PIERS
1b	
2b	
4b	−1166
2b	−2310
	+1330
2b	+217
4b	− 210
2b	+ 414
4b	+132
2b	− 204
4b	−128
2b	+ 510
4b	+129
2b	+ 310
4b	− 95
2b	+ 48
	+ 59
Σ	− 7
	−2095

FIG. 65.—COMPUTATIONS FOR A CONTINUOUS SERIES OF ARCHES SUPPORTED ON LONG SLENDER PIERS.

The table or form used in distributing the fixed-end horizontal forces and moments really consists of two parts, the one on the left being for horizontal forces and the one on the right for moments. The fixed-end forces and moments due to the load, P, in Span CD, are written in the top line (Steps 1(a) and 1(b)).

The unbalanced horizontal force, 65.5, at Joint C, is distributed as follows: The proportion to each arch $= \frac{J_a}{2J_a + J_p} = 0.128$; and the proportion to the pier $= \frac{J_p}{2J_a + J_p} = 0.744$. Using these distributing factors, a force of 8.4 goes to each arch and a force of 48.7 to the pier. Arrows have been used instead of plus and minus signs for indicating the direction of the forces.

The unbalanced moment, $-1\,440$, at Joint C, is distributed in the following manner: The proportion to each arch $= \frac{N_a}{2N_a + N_p} = 0.095$; and the proportion to the pier $= \frac{N_p}{2N_a + N_p} = 0.810$. From these factors, a moment of 137 goes to each arch and a moment of 1 166 to the pier. The signs are those required to balance the joint.

The remainder of the analysis is carried out as described under Steps 1 to 5 and the steps in Fig. 65 are numbered accordingly in order to facilitate reference to the outline. In distributing forces and moments to the piers, Steps 3 and 5 are not required, as the piers are fixed at the base.

When the distribution has been carried out to the desired degree of accuracy, the forces and moments at the ends of the arches and tops of the piers are obtained by adding the values indicated in Fig. 65. Knowing these, the stresses at any point can be determined by applying the laws of statics.

Value and Limitations of Method.—Three statements can be made as to the value and limitations of the foregoing method:

(1) As outlined it applies only to structures made up entirely of symmetrical arches.

(2) When the piers are relatively long and flexible, a number of distributions are required to obtain accurate results. Such piers are uncommon in the United States. When the method is used for the analysis of a multiple-span arch with the usual sturdy piers, several distributions will be sufficient and accurate results are quickly obtained.

(3) The greatest merit of this method lies in the ease with which the signs of all moments and forces may be visualized. Each step of the analysis has a definite physical meaning and the signs can be written by inspection, thus avoiding the mistakes in signs which are so inevitable in all methods which require the solution of simultaneous equations.

The procedure which has been described may be varied in a number of ways. The intent of the discussion has been to illustrate one way in which the general method of moment and force distribution may be applied to the analysis of multiple-span arches, or similar structures. The steps outlined

need not be considered as a "hard- and-fast" set of rules which must always be applied in the exact order described. The idea of using arrows instead of plus and minus signs to indicate the direction of the horizontal forces was suggested to the writer by Professor Cross, and experience has proved this to be a valuable expedient as it decreases the probability of errors in the signs.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

HIGHWAY LOCATION: GENERAL CONSIDERATIONS

Discussion

BY GIBB GILCHRIST, M. AM. SOC. C. E.

GIBB GILCHRIST,[5] M. AM. SOC. C. E. (by letter).[5a]—That serious mistakes have been made by following the principle of the majority rule is the opinion expressed by Mr. Mullen, who also states that these have later been condemned by the selfsame majority. This theory would be entirely correct if an engineer should build a structure of inferior and inadequate design, such as an important river bridge which might fail under increased loads, but in the case of highway location the engineer's duty should be to acquaint the governing body—whether it be the board of directors or the entire electorate —with all the facts and arguments for proper location, so that a principle of location may be established along which he may exercise the best technical knowledge for the most economical construction. He will find in a great majority of cases that, when the facts are presented by an engineer impartially and correctly, the decision as to policy will be along the proper line.

Mr. Harrison believes that too little emphasis is given to safety and he discusses the tendency of engineers to build a right-angle crossing of a stream with too much curvature at each end. Just as much criticism may be given to a plan in which alignment only is considered, and the bridge is constructed on too much of a skew. Latitude in this case should be given each way as easy curvature is not particularly objectionable and a compromise can easily be found.

The writer cannot agree with Mr. Harrison that the tendency is to do no more than is necessary "to get by." It is believed that engineers are giving more and more attention to location in a common-sense way and that radicalism in one direction is perhaps as bad as carelessness or vacillation

NOTE.—This paper by Gibb Gilchrist, M. Am. Soc. C. E., was presented at the Joint Meeting of the Highway and Construction Divisions, Dallas, Tex., April 25, 1929, and published in November, 1930, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: February, 1931, by Messrs. C. S. Mullen and H. L. Thackwell; and October, 1931, by Messrs. B. P. Harrison, and E. P. Arneson.

[5] State Highway Engr., Austin, Tex.

[5a] Received by the Secretary January 4, 1932.

in another. He cannot agree, either, with Mr. Harrison that more money should be spent to make highways "fool-proof"; this is an impossibility as far as location or construction is concerned. The only way to make highways "fool-proof" is to keep the "fools" off the highways, and the people riding in automobiles with "fools" as drivers will simply have to take their chances. Every consideration, of course, should be given for the safety of highways along reasonable lines for the protection of the traveling public.

Public sentiment has changed wonderfully on highway location, and it is still changing. It had evidently taken a step backward in the not too distant past, if evidence[9] given by Mr. James Dean, Land Agent and Civil Engineer, before a Committee on Highways of the House of Commons, on April 1, 1819, can be given credence. This might have been written in 1932:

"The first and most obvious improvement is to shorten distances; but even that must be governed by circumstances often of a local nature; a sound foundation, and the contiguity of good stone or gravel to a road, should not be overlooked in choosing a new line, or departing from an old one."

Reference was made in the original paper to a principle of democracy. Two letters written by President Thomas Jefferson under date of August 6, 1808, indicate that problems have not changed greatly in the meantime. They also show that the attitude of those in authority was not much different 125 years ago.

Writing to Kerr, Moore, and Williams, Commissioners of the "Western Road," Jefferson called attention[10] to a petition by the citizens of the Town of Washington (then in Pennsylvania) for a relocation of a through highway so that it would pass through their town. The argument was advanced that such a detour would be only a mile longer, that it would pass through better ground, and would cost less. While the principle purpose of the road was a communication directly westward, President Jefferson decided that if minor deflections would benefit particular places and accommodate travelers better, these circumstances should be considered. The letter ends by charging the Commissioners with the duty of reporting material facts and their own opinions, for the President's consideration.

On the same day President Jefferson wrote[11] to the Secretary of the Treasury, Albert Gallatin, and explained some of the location problems behind the letter to the Commissioners. It developed that this was a move made after a first concession had been granted to another town; a third petition was being prepared at still another town.

"I should have listened to these solicitations with more patience," wrote Jefferson, "had it not been for the unworthy motives presented to influence me by some of those interested. Sometimes an opposition by force was held up, sometimes electioneering effects, as if I were to barter away, on such motives, a public trust committed to me for a different object."

While many States are still using county aid obtained from the *ad valorem* tax for the construction of State highways, the tendency is not only for the

[9] "Road Making," by McAdam, p. 183.
[10] "The Writings of Thomas Jefferson," Vol. XII, p. 117.
[11] *Loc. cit.*, pp. 118–119.

construction funds to be obtained from transportation, but also for transportation to bear the assumption or refund of county funds spent on highway construction. For this and other reasons highway location should be determined according to the needs of the traveling public and the safety and convenience with which a traveler, or a group of transportation units, may reach their destinations with the minimum of travel and the maximum of safety, decisions being at all times governed by common sense for which a suitable substitute has never been found.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

SURVEYS FOR SWIFT RIVER RESERVOIR OF THE BOSTON METROPOLITAN WATER SUPPLY

Discussion

BY N. LEROY HAMMOND, M. AM. SOC. C. E.

N. LEROY HAMMOND,[7] M. AM. SOC. C. E. (by letter).[7a]—For surveys of the extent covered by the Swift River Reservoir Project, Professor Breed has well expressed the necessity of properly applying the yardstick to methods to be followed when he writes, "it is very easy to spend too much money on unimportant parts of the survey, and too little on essentials."

Professor Breed correctly states that greater accuracy in triangulation could have been obtained by a 30″ or 20″ transit, rather than by the 1′ transit actually used. On this particular project, instruments were selected which would be satisfactory for all phases of the entire project, including all necessary surveys and construction layouts. Because of the numerous checks afforded by existing State triangulation points, it was decided that the 1′ transit for triangulation would give satisfactory results.

While distances were computed in the triangulation system from the geographical positions of the State monuments used, it was deemed necessary to have an independent base line measured, in order to make certain that no error had been made in the selection of the older points. Had entire dependence been placed on the base-line measurement, it is probably true that the use of an invar tape would have been better.

The writer is in complete agreement with Professor Breed as to the possible use of the plane-table in filling in topographic features, and he believes that much of the Swift River Reservoir work could have been done with economy by the plane-table about as described by him. The writer has completed many surveys by this method, but feels that the make-up of personnel has much to do with its economical application. It is to be regretted that

NOTE.—The paper by N. LeR. Hammond, M. Am. Soc. C. E., was presented at the meeting of the Surveying and Mapping Division, Boston, Mass., October 10, 1929, and published in February, 1931, *Proceedings*. Discussion on the paper has appeared in *Proceedings* as follows: May, 1931, by Messrs. C. B. Breed and C. L. Hall; and November, 1931, by Frank C. Sellnow, Assoc. M. Am. Soc. C. E.

[7] Div. Engr., Metropolitan Dist. Water Supply Comm., Enfield, Mass.

[7a] Received by the Secretary January 20, 1932.

more engineers are not properly acquainted with the plane-table. In one survey with which the writer was connected, the party had started its detail by ordinary stadia methods when it was changed to the plane-table. After a week, the party chief was convinced that better progress could be made by the previous stadia method, but after a month he and his party were completely converted to the plane-table method over the ordinary stadia methods for the particular problem in hand.

Major Hall has indicated that the use of the true azimuth in a system of plane co-ordinates would involve considerable work for adjustment from point to point. The writer's statement in his paper that all lines were run on a true azimuth was incorrect, as lines were actually run from the grid azimuth derived from the true azimuth at the origin of the grid system. Major Hall has very clearly stated the advantage of this system.

In his discussion Mr. Sellnow has shown a broad vision of the general problem of reservoir survey. He has emphasized the necessity of properly tying in the origin of the local system of plane co-ordinates in order that the entire system may be referenced at any time to future use and extended mapping. In the Swift River Reservoir work, tie was made by a citation of the geographical position of the origin of the system.

If the control is well established with an accuracy of 1 part in 20 000, as cited by Mr. Sellnow, it appears to the writer that no hard-and-fast general rules can be fixed as to the accuracy to be secured in the details of the survey, but that value of real estate, capacity of reservoir basin, construction layout details, and other local factors influence largely the degree of precision which may be deemed necessary and sufficient for each survey, and that the engineer must decide upon his degree of precision as local conditions warrant, in order that the desired results may be secured with the greatest economy.

The aerial surveys of the Swift River Reservoir area were run in 1926, and it is probable that considerable advance has been made in this art since that time. The writer found variations as great as 25% in the scale of different portions of individual contact prints due to difference in elevation of ground surfaces and probably to a tilting of the camera by gusts of wind, etc. An effort was made to secure enlargements of contact prints to a scale of 1 in. equals 400 ft., but it was deemed more economical to make enlargements by means of a pantograph from the contact prints direct, and use the results as a basis of comparison with ground surveys. This decision may have been brought about by a lack of knowledge of proper methods to be adopted in enlarging contact prints.

While companies engaged in aerial photography may guarantee an accuracy, as stated by Mr. Sellnow, it appears to the writer that this guaranty must depend upon the ground control, and, in cases similar to the Swift River Reservoir area, when such ground control is secured, the use of the aerial survey has passed and necessary ground surveys have become well established. The writer, however, is still of the opinion that the aerial survey is a decided asset to the work of the ground survey and its expense is well justified in many ways.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FLOW OF GROUND-WATER AS APPLIED TO DRAINAGE WELLS

Discussion

BY M. R. LEWIS, M. AM. SOC. C. E.

M. R. LEWIS,[13] M. AM. SOC. C. E. (by letter).[13a]—The writer is fully aware that, as Mr. Hall points out, the formulas developed in his paper are not easily applied in advance of well-drilling operations. The chief value of such theoretical studies is in securing a better understanding of the laws governing the flow of underground water. Actual drilling operations must be governed by experience and judgment. However, judgment may be improved by a better knowledge of the underlying laws.

It is quite true that underground conditions are extremely variable, and that such studies might be extended to cover many other typical cases.

Mr. Hall states that the general opinion seems to be that, under similar conditions, Type *C* wells have a smaller draw-down than either Type *A* or Type *B* wells. It occurs to the writer that this opinion is perhaps based on observations of wells with faulty screens. The formulas developed by the writer did not attempt to take into account the loss of head through screens. In the case of fine sands which are sometimes screened out by the use of fine screens, the loss of head at the screen may be very large in Type *A* or Type *B* wells. On the other hand, in Type *C* wells a large quantity of the fine sand may be pumped out of the well, thus making its effective radius comparatively large.

It is true, of course, that wells pumping from deep-seated aquifers would not be effective for drainage if these strata were separated from the uppermost water-bearing material by a truly impervious stratum. However, experience has shown that in many instances such wells are effective, even where they appear to penetrate almost impervious strata. The explanation probably is that the rate of movement downward through the comparatively impervious strata, while slow, is yet appreciable. When the very large area over which vertical downward movement may be taking place, and the gradient causing the flow, are considered, the effectiveness of such wells is readily explained.

NOTE.—The paper by M. R. Lewis, M. Am. Soc. C. E., was published in March, 1931, *Proceedings*. Discussion of this paper has appeared in *Proceedings*, as follows: In September, 1931, by L. Standish Hall, Assoc. M. Am. Soc. C. E.; and November, 1931, by O. W. Israelsen, M. Am. Soc. C. E.

[13] Irrig. and Drainage Engr., Oregon State Agricultural Coll., and Bureau of Agricultural Eng., U. S. Dept. of Agriculture, Corvallis, Ore.

[13a] Received by the Secretary January 23, 1932.

In the paper the writer stated a conclusion that successful drainage by means of wells requires a general lowering of the water-table, unless the area to be drained is only a few hundred feet in diameter. When a general lowering of the water-table is required, the location and design of wells would be based entirely on the economics of the problem, and wells should be located and drilled to such depth as will supply water most economically. The same problem of economics applies to the depth to which the water in the well should be drawn. There are many interesting problems in this connection. Mr. Hall suggests the use of a battery of shallow wells. In some instances such batteries of wells with very low draw-down in each well may be economical. As Mr. Hall points out, the question of cost of pumping equipment is then an important factor.

Professor Israelsen is correct in his statement that the term, "transmission constant," was used by Slichter rather than the term, "transmission coefficient." He is also correct in pointing out that the definition of the transmission constant is incomplete. Perhaps it was best stated by Slichter himself,[14] as follows:

"The constant, k, is the quantity of water that would be transmitted in unit time through a cylinder of the soil of unit length and unit cross-section under unit difference in head at the ends."

The transmission constant, k_2, suggested by Professor Israelsen, having the physical dimension of time, is numerically the same regardless of the units of force and length used in its determination. This is, unquestionably, an advantage as compared with the transmission constant, k, used by Slichter, the numerical value of which depends on the unit of length used. However, in both field and laboratory studies, units of force and length must be used. If k_2 is to be used, the appropriate value of g, the acceleration due to gravity, must be applied in determining the gradient. This fact seems to counterbalance the advantage of a constant having a single value regardless of the units of length used. Moreover, Slichter's k, or similar constants in other units, is almost universally used in the study of the flow of water through saturated materials. For these reasons the writer does not deem it wise to use the constant here represented by k_2.

As used in Equation (7), and except for the single condition that k_1 is the maximum limit of the value of w, the rate of percolation, w, and the transmission constant of the relatively impervious material, k_1, are independent constants.

As Professor Israelsen points out, the value of w is an important factor in influencing the effectiveness of drainage and the area which may be drained by a given well. It is quite probable that in the case of the Utah well mentioned, the value of w is extremely small. If this is true, the value of R would be high and, therefore, the radius of the area having a comparatively large draw-down would be large. Under such conditions a comparatively small number of wells would serve to lower the general water-table level over a large area.

[14] Nineteenth Annual Rept., U. S. Geological Survey, Pt. II, p. 323.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

GRAVITY DAMS ARCHED DOWN STREAM

Discussion

BY MESSRS. FLOYD A. NAGLER, AND B. J. LAMBERT

FLOYD A. NAGLER,[15] M. AM. SOC. C. E. (by letter).[15a]—The question raised by Professor Lambert with regard to the arching of gravity dams, is quite valid for dams built between narrow canyon walls with the abutments parallel to the canyon wall itself. However, the abutments of such dams would generally be seated into the canyon wall, and the dam would no longer act strictly as a gravity dam. In all such cases it would be preferable to arch the dam up stream rather than down stream, in spite of the considerations to the contrary for the true gravity type, which have been presented by the author.

Regardless of the fact that gravity dams are generally analyzed in linear-foot sections one should never fail to consider the behavior of the dam as a whole. This has received too meager emphasis in most textbooks, although it is casually mentioned by some authors. Some engineers assert that a large dam does not act as a unit, as assumed by the author, but one must answer the question as to how large a dam must be before it will fail to do so. The small models described by Professor Lambert certainly acted as a unit, and if large dams do not do so, somewhere in the range of size of masonry dams, there must be a structure so large that no shearing stress exists between adjoining sections. This is absurd. Many gravity dams known to the writer still stand because they are acting as monoliths, whereas erosion has so seriously undercut certain sections that if considered by themselves they should have failed long ago. The evidence disclosed by the St. Francis Dam disaster indicates that the shearing stress between adjacent sections of the dam must have been exceedingly great. Although gravity dams have numerous construction and expansion joints, interlocking generally exists so that the dam can function more or less as a unit. Hence, the author's contentions should stimulate serious consideration.

NOTE.—The paper by B. J. Lambert, M. Am. Soc. C. E. was published in March, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: In August, 1931, by Messrs. Eugene Kalman, William P. Creager, and T. L. E. Haug; and October, 1932, by Messrs. B. F. Jacobsen, and A. Floris.

[15] Prof., Hydr. Eng., Univ. of Iowa, Iowa City, Iowa.

[15a] Received by the Secretary November 2, 1931.

B. J. LAMBERT,[16] M. AM. SOC. C. E. (by letter).[16a]—The author would like to make it clear that he believes that a gravity dam arched down stream can be used only in a canyon having particular features as to width of canyon, slope of walls, and height of dam. This might occur only once in a hundred or more locations, so the discussion is more or less academic. This scheme would be ridiculous if it were applied to the average canyon dam, regardless of the factors mentioned.

There also seems to be a difference of opinion as to the size of dam that ceases to act as a unit and begins to act in sections. It is true that English-speaking engineers have become accustomed, in using the pressure formula, $\frac{wh^2}{2}$, to define w as pounds per cubic foot of water and h, in feet, thus implying slices 1 ft. in width; but in the metric system, as used in France, Germany, Spain, etc., the terms, w and h, are based on the meter, in which the volume is about thirty-five times as great as that based on the foot. It is not unreasonable to suppose that the "Brobdingnagians" used a linear unit equal to ten or even one hundred times that of American units, with a corresponding volume equal to 1 000 to 1 000 000 cu. ft.

It would seem almost obvious that the sizes of the blocks into which a dam might break is a function of the thickness of the dam. If the average thickness is a few inches or a few feet, the sizes of the blocks might well be a few cubic inches or a few cubic feet; but when dealing with dams several hundred feet thick it seems reasonable to assume that the blocks or units that might be considered, would be of a much larger volume.

Where the back of a dam is constantly covered with water to a greater or less extent, the temperature changes may not be extreme through a large portion of the structure, so that the breaking into blocks is more or less limited by the shrinkage incident to the setting up of the concrete. If these contractions are under fair control, as will be attempted in the Hoover Dam, it is reasonable to suggest that possibly this dam can act practically as a unit, since it will have a maximum thickness of 650 ft., with an average of, say, 300 ft., or more, an average length of about 500 ft., and a maximum height of approximately 650 ft. If this huge plug of concrete were constructed in a canyon with smooth sloping walls, as shown in Fig. 2(*a*), there would be little doubt in the writer's mind, that the stresses on the smaller length of toe would be much in excess of those computed by the ordinary slice method. However, when one considers that, in practice, the sloping walls are roughened and also made more or less normal to the radius of the arch plan, then these modified conditions make the problem so complex that even an approximately exact analysis is impossible.

It is not difficult to imagine various things that might be occurring in a structure where such immense masses, pressures, and uncertain conditions are involved, therefore, in presenting this brief paper, it was the writer's intention to set out merely another point of view, the merits of which may be dependent on many debatable factors.

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[16] Head, Dept. of Civ Eng., State Univ. of Iowa, Iowa City, Iowa.

[16a] Received by the Secretary January 22, 1932.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

AN ANALYSIS OF MULTIPLE-SKEW ARCHES ON ELASTIC PIERS

Discussion

BY MESSRS. RICHARD M. HODGES, AND J. CHARLES RATHBUN

RICHARD M. HODGES,[36] ASSOC. M. AM. SOC. C. E. (by letter).[36a]—In Part IV of his paper, Professor Rathbun gives a simple and direct method of designing skewed arches after the forces and moments acting on any section have been found. In the writer's opinion this is the most valuable information, from a strictly practical standpoint, contained in the paper. It is also, probably, the only part of the paper that may be considered open to question.

As far as all external forces and reactions are concerned, the solution of skewed arches, whether fixed or hinged, single-spanned or multi-spanned, is no longer a matter of opinion. The correctness of Professor Rathbun's results has been verified by tests, and in the case of hinged skewed arches of one span, at least, has been checked by an independent analytical procedure,[37] as well as by the investigations of B. L. Weiner,[38] Assoc. M. Am. Soc. C. E.

However, when it comes to the matter of finding the distribution of stresses on any section, and of designing the arch and its reinforcement to withstand these stresses safely and economically, no such certainty exists. It seems evident that, whatever method is used, an exact design will be impossible until tests have been made to verify the fundamental assumptions of stress distribution which must be made at the outset, the correct value of the constant, K, in the torsion formula, $v = \frac{Kf}{bd^2}$ (and the applicability of the formula itself to skewed arches), and the practical effectiveness of whatever method of torsion reinforcement may be proposed. Pending such tests, it will remain

NOTE.—The paper by J. Charles Rathbun, M. Am. Soc. C. E., was published in April, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1931, by Messrs. B. L. Weiner and G. D. Houtman; and November, 1931, by Messrs. Charles L. Nord, F. W. Keyser, and A. A. Evenin.

[36] Asst. Engr., Westchester County Park Comm., Long Island City, N. Y.

[36a] Received by the Secretary December 10, 1931.

[37] "The Rigid Frame Bridge," by A. G. Hayden, M. Am. Soc. C. E., N. Y., John Wiley & Sons, 1931.

[38] *Proceedings*, Am. Soc. C. E., January, 1931, pp. 1–20.

a matter of opinion as to what degree of analytical precision is required in deriving design formulas, and as to what degree of reliance can be safely placed on such formulas in practical design. The writer is of the opinion that the author's method, with the modification hereinafter suggested, is the best practical method yet proposed for designing skew arches, on account of its directness and simplicity, and because it leads to results which seem safe and reasonable for all ordinary skews.

Luckily, after all, indications point to the probability that for moderate skews, the old method of designing a skewed arch as a rectangular arch for a span equal to the skewed span is fairly safe. In the writer's opinion, the author's method is applicable to skews up to 45°, more or less, which comprises the great majority of grade-crossing eliminations. Skews of more than 50° should be avoided, pending the results of tests.

In applying the author's method of design, the following modification is suggested. It is often found advisable from the standpoint of practical simplicity to place the transverse reinforcement parallel to the abutments. In such a case, the forces and moments, being resolved into directions parallel to spandrels and parallel to abutments, become:

$$t = t_v \sec \epsilon' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (53)$$

$$s = t_v \tan \epsilon' - t_z \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54)$$

$$m = m_z \sec \epsilon' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (55)$$

and,

$$f = m_z \tan \epsilon' + m_v \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (56)$$

The remaining procedure then follows, as shown in the paper. This change seems to result usually in an increase in the longitudinal reinforcement, but sometimes a decrease, depending on the relative magnitudes of t_v and t_z, or m_z and m_v. Wherever the resultant, r (see Fig. 8), is in the exact direction of the arch axis, Equations (53), (54), (55), and (56) will lead to the same results as the author's formulas. It is suggested that, in practice, the design should be made in every case according to the formulas leading to the larger amount of longitudinal reinforcement regardless of the selected direction of the transverse steel. The increase in cost will be inconsiderable in comparison with the increased margin of safety.

The design of the transverse reinforcement becomes a matter of academic interest only, since the resulting steel is nearly always less than the transverse spacer rods which would be inserted in any case to tie the longitudinal steel. Inadvertently the author seems to have equated and unbalanced shear torque to an unbalanced resisting steel stress couple in order to find the required steel areas. A more correct procedure would seem to be to take moments first on one side of the neutral axis and then on the other. However, this correction leads to very small variations, in application, from the results obtained by doing it the author's way.

J. CHARLES RATHBUN,[39] M. AM. SOC. C. E. (by letter).[39a]—Some of the points that have been brought out by those who have contributed to this paper are pertinent to the subject of arches in general rather than to the effect of the skew. As they add to the knowledge of arches and arch design the writer will give his contribution as well as discuss those points that arise in the design of skew arches only.

Mr. Weiner indicates that a set of deflection formulas should be worked out for the case of a multiple-skew arch hinged at the base. The paper contains, in Appendix III, the formulas for the skew arch when the abutments are free to rotate, as he suggests. The body of the paper gives the general method of extending the analysis from one to two or more spans. The writer feels that this constitutes a solution of the problem even if an example of application is not given. The extension of these formulas to include the multiple arch involves no new ideas that have not been brought out in this paper, in the writer's previous paper,[39b] and in Appendix III. Working out and publishing these formulas would have involved considerable labor and would have lengthened the paper materially. The writer suggests to any one who may have use for these formulas and may wish to derive them that he divide the structure by cutting planes as shown in Fig. 32.

FIG. 32

The local load distribution due to a concentrated load, as demonstrated by tests made by Mr. E. F. Gifford (Figs. 16 to 25, inclusive),[39c] shows that dangerous local stresses surround the point of application of this load. This very important problem arises in the design of arch rings where the load covers a comparatively restricted area and will arise in right, as well as in skew, arches.

The writer conducted a simple experiment which may add to the findings of Mr. Gifford. It also gives some idea of the lateral distribution over the ring of a concentrated load. Fig. 33 gives the general set-up of the experiment. A sheet of galvanized iron about 20 in. wide was bent to form an arch with about 5 in. rise and a span of $13\frac{1}{4}$ in. The ring was a segment of a circular cylinder, quite wide and very thin. The ends may be considered fixed, because the sheet extended about 4 in. beyond each abutment. This 4-in. extension was placed under a 1 by 4-in. plank which, in turn, was nailed to the table. A concentrated load was applied by weighting a 60d

[39] Associate Prof., Civ. Eng., Coll. of the City of New York, New York, N. Y.

[39a] Received by the Secretary February 1, 1932.

[39b] "Analysis of the Stresses in the Ring of a Concrete Skew Arch," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 611.

[39c] *Proceedings*, Am. Soc. C. E., September, 1931, p. 1109.

nail which had been driven through a plank, as shown in Fig. 33. Weights were placed on this plank, but no attempt was made to evaluate the load.

A central load was placed on the arch and the deflections along the crown from face to face were noted by means of an Ames dial placed on a carrier. There was some permanent set, as shown, when the load was removed. The

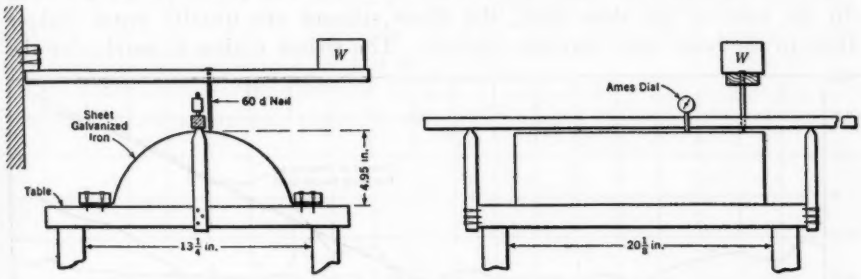

FIG. 33

load was then placed near one edge of the crown and another set of deflections taken. The deflection and permanent set due to the second load were taken from the final position after the removal of the first load.

Fig. 34 shows three curves for each load, first the deflection along the crown of the arch while the load is in position; second, the position after the load is removed (the permanent set); and, third, the difference between the two, which gives the distance that the crown rose after the load was removed. These curves show that there is considerable deviation from a straight line near the load. This simple experiment gives some emphasis to Mr. Gifford's work. A practical as well as a theoretical solution of this problem would be very valuable, but until it is made it is evident that designers should reinforce structures so as to avoid failures of the type indicated by Figs. 16 to 25.

Mr. Weiner takes issue with the writer because he does not use the ellipse of stress, and states that his Equations (7) "are merely a transformation of axes." This is evidently a misunderstanding because the writer has not transformed axes, as is done in the common method of deriving the ellipse of stress. The writer computes the components of a force and from them he finds that force, which is then broken up into other components that are more convenient to use in obtaining unit stress. The ellipse of stress that is found by considering these forces may have one axis equal to zero; that is, it may be a straight line. The properties of such an ellipse do not contribute to the understanding of the problem, as the writer sees it, and, therefore, he has not used it in his solution.

To make this point clear the stresses on a point near the edge of the ring will be considered. The line of force, or direction of force, must be parallel to the surface when it is a very short distance from this surface. This means that there is no shear across the plane perpendicular to the force and it is, therefore, one axis of the ellipse. The force perpendicular to it (if it is of any value) will be perpendicular to a surface. It is therefore

zero, and the ellipse is a straight line. This is the point used by Mr. Weiner in the illustration in his discussion.

The writer feels that the problem of the ellipse of stress in the skew arch is somewhat the same as that in a reinforced concrete beam. It is interesting to study these ellipses at various points in a beam, but a designer who is proportioning stirrups does not consider this phase of the problem. In the case of the skew arch, the shear stresses are usually much lighter than in the beam that requires stirrups. The writer wishes to emphasize the

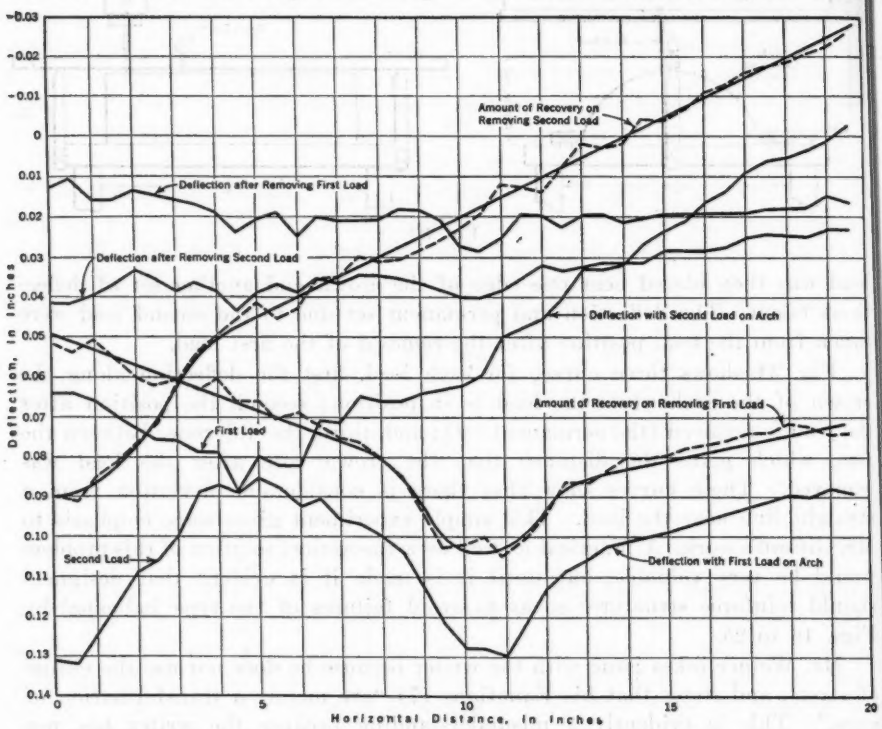

FIG. 34.—DEFLECTION ALONG THE CROWN OF A WROUGHT-IRON SEGMENTAL ARCH UNDER CONCENTRATED CROWN LOADS

fact that although T_x, T_z, and T_y are evaluated separately, they are only a part of the force, T. Combining components of a force to find the force is only one step in what Mr. Weiner refers to as the first part of the skew-arch problem.

Mr. Weiner takes issue with the writer on the method of computing unit stresses and shows by an example the difference between the results obtained by his method and that presented in this paper. He states that the writer is in error in claiming that the two methods are the same, which is admitted. As Mr. Weiner states, the one given by the writer has been used for several years and until carefully conducted laboratory tests or other demonstration proves the superiority of another method, he will consider it as close an approximation as has been found. The assumptions have been stated and

are quite clear, the entire method is easy to understand and simple to apply. Mr. Weiner has presented a method which is based on assumptions of the same general character, slightly different from those given by the writer, but no more logical, and without proof. As his method has no better foundation than that of the writer and leads to a result with a shear on one plane and a thrust on another at each point, it is difficult to understand why the change has been made. Like Mr. Weiner, the writer does not consider the effect of the curvature of the arch ring.

One way of comparing the two methods is as follows: The writer assumes uniform thrust across the plane, *hi* (Fig. 35), in the direction parallel to the

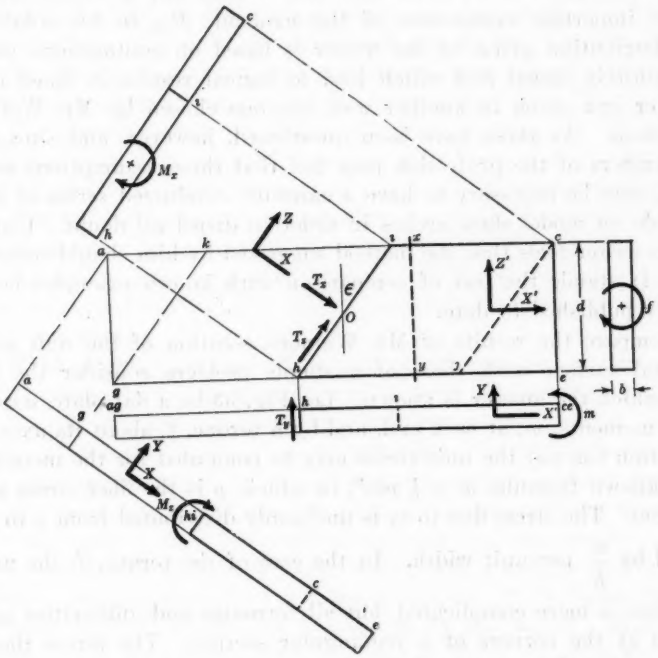

FIG. 35

X'-axis (in the direction of the major line of reinforcement), and a shear parabolically distributed at right angles to it or parallel to the Z'-axis. Mr. Weiner assumes uniform thrust across the plane, *hi*, in the direction of the X-axis. From this assumption he deduces the writer's assumption as to thrust. He also assumes a shear parabolically distributed in the direction of the Z'-axis, on the *hi* plane. He then has at each point a thrust in the direction of the X'-axis across a plane parallel to the Z'-axis and a shear (along a different plane), parallel to the Z-axis. This shear has a component parallel to the line of thrust, the X' direction, which makes the thrust non-uniformly distributed although the thrust plus the component of the shear are so distributed. The writer considers the thrust and shear perpendicular to each other and thus avoids this phase of the problem.

Comparing the internal work of the shear and thrust the writer has uniform thrust against non-uniform thrust by Mr. Weiner, the former giving the less work. Both have parabolic shear at right angles to this thrust. Therefore, the writer's method assumes less internal work then that of Mr. Weiner. This is not a proof of the correctness of the assumptions, but does show they are more nearly correct than those submitted as a substitute.

In the problem chosen in the discussion by Mr. Weiner his method gives a result 25% in error and, therefore, the writer considers that the statement that "his method is sufficiently accurate for design purposes" is questionable. The writer feels that the assumptions made in his paper do not lead to such large errors. It is to be noted that Mr. Weiner did not take into account the very important component of the moment, M_x, in his solution. The stress distribution given by the writer is based on assumptions which have been definitely stated and which lead to logical results in those cases that the writer can check in another way, the one chosen by Mr. Weiner being one of them. As these have been questioned, however, and since, perhaps, some members of the profession may feel that these assumptions are unwarranted, it may be necessary to have a carefully conducted series of laboratory tests made on model skew arches in order to dispel all doubt. Until this is done, the writer feels that the method suggested by him should serve for arch design. It stands the test of comparison with known examples better than any other published to date.

To compare the results of Mr. Weiner's solution of the unit stresses on a diagonal surface with those of a simple problem consider the following case, in which the answer is known. Let Fig. 35 be a flat plate, *a c e g*, acted upon by moments, m, at each end, and by a torque, f, about its axis. At any right section (as xy) the unit stress may be computed for the moment, m, by the well-known formula, $m = \frac{1}{6} pbd^2$, in which, p is the fiber stress at the top and bottom. The stress due to m is uniformly distributed from x to y, and is expressed by $\frac{m}{b}$ per unit width. In the case of the torque, f, the unit stress distribution is more complicated, but all formulas and authorities agree that it is zero at the corners of a rectangular section. The stress then at the points, x, y, i, and h, or any other point on a corner, is that due to a moment of $\frac{m}{b}$ and is in a direction parallel to the corner line. This is true for all values of f. The force across a plane parallel to and near the vertical face, ac, is zero. The ellipse of stress has degenerated into a straight line. If this flat plate is cut by an imaginary plane, hi, making an angle, ε, with the right section, and if the portion to the right of the section is removed, it must be replaced by the moment and force produced by the removed portion. This moment may be broken up into three components, M_z, M_x, and M_y, while the force is broken up into T_z, T_x, and T_y. Taking the forces and moments about each axis equal to zero six equations of equilibrium are evolved; $M_y = 0$; $T_x = T_y = T_z = 0$; while $M_x \cos \varepsilon + M_z \sin \varepsilon = f$; and, $M_z \cos \varepsilon - M_x \sin \varepsilon = m$.

If M_x and M_z are known, m and f may be found and the unit stresses computed. As m may be taken as uniformly distributed at the ends of the plate the stress at i must be $\frac{m}{b}$ and in the direction of ac no matter what the value of f. Taking the case chosen by Mr. Weiner in which $M_x = -361.9$; $M_z = 533.57$; $\tan \epsilon = 1.079$, and $d = 31$ ft. By computation, $m = 628.1$, and the unit stress at i and h is that produced by a moment of $\frac{628.1}{31}$, or 20.2 kip-ft. per lin. ft., and not 25.3, as reported by Mr. Weiner. The writer, therefore, does not wish to endorse Mr. Weiner's method.

Mr. Eremin and Mr. Hodges bring up the question of the reinforcement in the steel which is placed at an angle with the force. It is interesting to compare Mr. Eremin's equations with those given by Mr. Weiner[40] in Equation (60) of his paper "Design of a Reinforced Concrete Skew Arch," noting that his δ_2 is zero when the ellipse of stress is only a straight line. The shear induced in the concrete by the steel placed at an angle tends further to reduce the load on the steel. In most cases the direction of the force follows the direction of the steel very closely, and the writer questions whether there is great error in the customary method of reinforcing for the parallel component of the force and then reinforcing for shear with bars acting somewhat as stirrups do in a beam. Of course, in extreme cases this point should be investigated.

Mr. Nord states that the direction of stresses in a skew arch are "apparently contrary to the fundamental law of Nature that work is done by forces along the shortest lines and in the most economical way." The latter part of the statement is most important, the former being assumed, at times incorrectly, from it. It is just this assumption that has made it difficult for engineers to believe the results of tests and the theory in the case of the skew arch. The equations used in this theory have been checked by the "method of least work" by the writer and by others, including Mr. Keyser. Such solutions as the one suggested by Professor Oskar Muy involve more internal work than that given by the writer. Although they under-estimate or neglect the force, T_z, and so make the force at the crown assume a direction nearer the obtuse corner, the resulting lateral bending of the arch produces more internal work than is gained by reducing the distance traveled by this force.

Mr. Houtman's discussion brings out quite clearly the fallacy of this line of thought. If an arch is divided into parts by a cutting plane all the forces across this plane should be considered. This includes thrust. Those who divide the arch by planes perpendicular to the abutment into two triangles and a rectangle (as viewed in plan) do not evaluate these forces and moments; nor do they even estimate this thrust. At least, the writer has known of no case in which the analysis has been carried through.

As Mr. Nord suggests, certain terms may be eliminated in the practical design of an arch. His Table 6 should be interesting and valuable as a ref-

[40] *Proceedings*, Am. Soc. C. E., January, 1931, p. 54.

erence to one who has an arch to analyze of the type studied by Mr. Nord. He can there see the relative value of the various terms.

Mr. Keyser calls attention to the fact that the stresses due to eccentric loads on a right arch can be worked out by the equations given by the writer. In this connection the local stresses caused by a concentrated load, as shown by Mr. Gifford's experiments, must be considered in the final design. Mr. Keyser's check of the writer's equations by an entirely independent method—the method of virtual work—has been very valuable and encouraging. His suggestions about foundation movement can well be enlarged upon. If the movement of a foundation is anticipated it may be assumed that it is proportional to the load, or that it is a definite amount. In the latter case the deflection at the crown section can be computed. Since the load terms are computed as deflections of this same section this added deflection can be used as a load term and the stresses can be found in the usual way. If the deflection of the abutment is assumed to be a function of the abutment load the pier may be assumed to be extended, the extension being of such dimensions that it will have the same elastic properties as those assumed for the soil. The wisdom of building an indeterminate structure on yielding foundations is questionable, unless due consideration is taken of the effect of the yielding; so Mr. Keyser's warning is a wise one.

Mr. Eremin suggests several simplifications which will doubtless shorten the work in the analysis of those designs when they do not create serious error. It has been the writer's policy to develop the equations without neglecting small terms, in order that these equations may be used in types of design where they may be of considerable influence. It is a comparatively easy task to drop small terms, but difficult to supply them if dropped in the original development and afterward found to be of importance. Mr. Eremin suggests that arithmetical errors may accumulate, but this can be avoided if one uses a mechanical computation machine. It involves little more labor to carry the work out to enough decimal points to keep the error well within the range of the accuracy required.

Mr. Hodges is correct in his statement that the theory of the two-hinged arch has been checked as to the algebra of the analytical solution. This check was made by Mr. Weiner and H. G. Babcock, M. Am. Soc. C. E., under the direction of A. G. Hayden, M. Am. Soc. C. E. Mr. Hayden has given this theory in the book referred to by Mr. Hodge, but not as a check on the theory. Mr. Weiner states quite clearly in the "Introduction" to his paper,[41] that he is simply presenting this theory as the writer developed it, but has "revised the form of the equations." No changes or additions affecting the theory were claimed (except for unsymmetrical skews), but the changes were made in order to simplify or clarify the work. As brought out in the discussion of that paper the writer does not think any of the changes an improvement and some were unwise because they were not consistent with accepted usage in mathematics, analytics, and the beam theory, and because they tend to confuse.

[41] *Proceedings*, Am. Soc. C. E., January, 1931, p. 4.

The writer questions the limitation of 45° placed by Mr. Hodges on the skew-arch theory. There may be some question as to the assumptions of the writer in calculating the unit stress, and Mr. Weiner has shown that other assumptions can be made that lead to different results. However, the general theory has been definitely corroborated. As Mr. Hodges states "the transverse reinforcement becomes a matter of academic interest only" and so this matter of the difference in the two methods of computing unit stress is not as great as might be supposed at first as far as transverse reinforcement is concerned.

In the case of unbalanced torque the writer has simply attempted to place steel in the form of stirrups that is an equivalent for the unbalanced concrete, as shown in the shaded triangle of Fig. 9. This steel must be an equivalent in moment as well as in thrust. As the force has a lever arm of 10.64 in. and the steel has only 8 in., the area must be changed accordingly. The direct thrust must also be checked if the change affects it, as it might where the lever arm of the steel is greater than that of the concrete.

The writer wishes to thank those who have contributed to this discussion and thus have helped to develop this subject of skew and multiple-skew arches.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

MANUFACTURING CONCRETE OF UNIFORM QUALITY

Discussion

BY JOHN H. GRIFFITH, M. AM. SOC. C. E.

JOHN H. GRIFFITH,[21] M. AM. SOC. C. E. (by letter).[21a]—The writer is interested in this paper because of the information afforded as to the range of variation to be expected in systematic analyses of concrete laid under first-class conditions of fabrication, control, and tests. He believes that a higher degree of efficiency could be obtained than is usual in the testing of concrete if the capping of cylinders could be replaced by grinding the ends of specimens to a true plane approximately normal to the axis of the cylinders. This is easy with a cast-iron disk mounted horizontally and rotating about 30 rev. per min. Ottawa sand is a good abrasive. A number of cylinders can be handled simultaneously with very little watching, the proper pressure being supplied by their own weights. Both brick and hollow building tile develop higher and more uniform strengths by this method than by the older methods of capping. As Mr. Little has remarked in Appendix I the operator cannot be sure of a plane surface in capping and by the use of an intermediate paper. As he rubs the cylinder on a steel plate in any case, he need only carry this a little further to get a perfect job once for all, and without rocking and other expedients.

The ideal theoretical method for mounting silicate specimens in compression is the hydraulic bearing originated by Mr. Mont Schuyler[22] formerly of the Municipal Laboratory at St. Louis, Mo. This method in principle consists in mounting the specimen on a hydrostatic base, eliminating entirely the uncertain and quite considerable radial friction developed under pressure by the concrete expanding radially under test. The common method of frictional reinforcement at the base produces a cone-shaped fracture, while the test under a pure compression gives vertical planes radiating from the axis

NOTE.—The paper by William M. Hall, M. Am. Soc. C. E., was published in May, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: September, 1931, by Messrs. Henry B. Seaman, Theodore Belzner, John Sanford Peck, and J. W. Kelly; November, 1931, by Messrs. Stanton Walker and Edward E. Bauer; and December, 1931, by Messrs. O. Bonney and D. T. Mitchell, C. E. Arnold, and Morris Mason.

[21] Prof. of Experimental Eng., Iowa State Coll., Ames, Iowa.

[21a] Received by the Secretary January 6, 1932.

[22] "Spherical Bearings," by Mont Schuyler, *Proceedings*, Am. Soc. for Testing Materials, Vol. XIII (1913), p. 1004.

of the specimen 120° apart. The method proposed by Mr. Schuyler needs mechanical development since there is the problem of a possible "blow-out" under the high pressures involved. While the surface grinding method has some radial friction, the writer believes that it will give efficient test results as high as or even higher than Mr. Schuyler's method as a result of the perfect square ends and because the vertical filaments are under a minimum of eccentricity, there being scarcely any incipient local failures by this method.

Rather frequently in testing practice use is made of hemi-spherical heads that are likely to "stick" as a result of gumminess and infrequent oiling. It is a good plan to give a hurried wiping and squirt of oil to these heads preliminary to every test so that they are sensitive to every eccentric load however slight. This plan avoids applying test loads at the peripheries of specimens, such that they lower the intrinsic strengths of concrete. Too often this danger is not realized in practical testing. One should seek to develop the full theoretical strength of a specimen even if it is difficult always to attain it in practice.

The following method has been found of general utility in making specific gravity, displacement, and absorption tests of the silicate products, including sand, rock, refractories, and concrete. It has the advantage of being performed either on a small or a large scale, with practically any sort of container, a pycnometer, an Ehrlenmeyer flask, a Schuette cone, a can with a constant level spout, or even a pail. Three-figure accuracy is possible with reasonable care, and four figures by careful work with temperature correction determinations. It is a method of differential weighing; the equations are as follows: Let the tare of the receptacle be represented by $W_0 = w_0$. The weight of the container filled with water to a mark or to the top equals,

$$W_1 = w_0 + w_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and the weight of the container, the water to the same mark, and the weight, S, of the specimen, equals,

$$W_2 = w_0 + w_2 + S \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Subtracting Equations (1) and (2),

$$W_2 - W_1 = S - (w_1 - w_2) = \Sigma\, w - \Sigma\, \frac{w}{t} = \Sigma\, w\left(\frac{t-1}{t}\right) \ldots\ldots (3)$$

in which, $\Sigma\, w = S$, is the weight of the specimen in terms of its separate components, if any, either intact or powdered in a mortar to suit, and sifted, and $\Sigma\, \frac{w}{t}$ is the summation of the displacements of components in terms of the specific gravities. The displacement, in grammes or cubic centimeters, of the specimen, $S = \Sigma\, w$, is $w_1 - w_2$.

In the tests of the silicate products, $\frac{t-1}{t}$ is approximately the fraction, $\frac{5}{8}$, and is to be chosen subject to the statistical experience of the operator or determined by sufficient check tests. Since the specific gravity of water

is ordinarily taken as unity, $\frac{t-1}{t}$ for water vanishes. Hence, there results (to a close degree of approximation):

$$\tfrac{8}{5}(W_2 - W_1) = \Sigma w = S_d \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

for the dry weight of the specimen, and for the absorption of water, $S_w - \frac{8}{5}(W_2 - W_1)$, the other elements of the specimen, if any, being found by sieve separation in the usual manner, dry or wet according to circumstances. The figures for a run of wet concrete will demonstrate its inherent advantages as an expeditious method, the process being still quicker in the cases of paving and building bricks, and refractories (see Table 5). A constant level container was used.

TABLE 5.—WEIGHT OF A MIX, APPROXIMATELY SOAKED, BUT DRY ON THE SURFACE

Item	Symbol	Weight, in kilogrammes	Description
1	W_0	2.216	Tare
2	W_1	12.990	Container and water
3	W_2	16.270	Container, water, and specimen
4	w_g	3.000	Gravel
5	w_s	1.500	Sand
6	w_c	0.750	Cement
7	w_w	0.375	Water
8	S_w	5.625	Wet mix

The difference between Items 3 and 2, Table 5, is 3.280 kg. Taking eight-fifths of this gives the dry weight of the mix as 5.248 kg. Subtracting this value from Item 8, Table 5, gives 0.377 kg. in contrast to 0.375 kg. for the original weighed water (which, of course, is frequently unknown in practice).

The specific gravity of Ottawa sand, using a 10 or 20-gramme sample in a pycnometer, can be found by the method mentioned herein so as to give the order, 2.65, as usually recorded with repetitions within the range, 0.01, sometimes recommended. For refractories and paving bricks it has been found that five samples can be tested in the space of a few minutes using a 10-gal. pail with a "fiduciary" pencil mark on the inner surface of the pail to give results of the orders published by the authorities. In the case of relatively non-absorbent materials like paving brick, scarcely any water can be absorbed in the short interval of the test. When absorption tests have been run, the true or apparent specific gravities are quickly found without pulverizing materials, which is sometimes a factor in hurried determinations.

In making routine tests it is advisable to obtain once for all, the values of W_0 and W_1 as averages from a number of determinations. Using an empty, dry receptacle the sample is then best weighed for the gross, and the water filled to the mark later to give W_2. In general, by the use of a strictly differential method with as few manipulative operations as possible high efficiencies are obtainable with few losses due to transfers and other handlings.

It is the purpose herein to give the writer's experience in testing rather than to criticize Mr. Hall's valuable paper on the difficult problem of handling a synthetic material.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ENGINEERING FEATURES OF THE ILLINOIS WATERWAY

Discussion

BY MESSRS. DAVID A. MOLITOR, AND CHARLES A. D. YOUNG

DAVID A. MOLITOR,[17] M. AM. SOC. C. E. (by letter).[17a]—In this paper, the author has attempted to describe briefly a work comprising a variety of structures entering into the construction of the Illinois Waterway, and he gives special consideration to the stress analysis of the lock-walls with large culvert openings, which he claims to have solved by applying the principle of least work. Several of the subjects mentioned, since they are not adequately treated in books, might properly cover more space than was allotted to the entire paper.

Mitering Lock-Gates.—In designing horizontal girders of mitering lock-gates, the chief difficulty is in determining appropriate flange areas for the case of direct stress and bending, where the direct stress, the bending moment, and the moment of inertia, all change at each girder section, as shown on the author's Fig. 6(*a*).

The writer has found the following formulas very useful in avoiding the customary trial-and-error method of proportioning the required girder flanges.

In addition to the notation used by the author, the following are introduced (see Fig. 17):

F = area of flanges in Fig. 17; F_1 refers to Flange No. 1 and F_2 refers to Flange No. 2.
f = unit stress; f_1 refers to extreme fiber of Flange No. 1 and f_2 refers to extreme fiber of Flange No. 2.
d = depth of web-plate.
t = thickness of web-plate.
j = lever arm of resultant, T.
T = resultant force acting on the girder section, A-A.

NOTE.—The paper by Walter M. Smith, M. Am. Soc. C. E., was presented at the meeting of the Waterways Division, Milwaukee, Wis., July 11, 1929, and published in October, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings* as follows: February, 1932, by Messrs. A. W. Sargent and E. G. Walker.

[17] Cons. Engr., Detroit, Mich.

[17a] Received by the Secretary December 18, 1931.

Case I.—To find the theoretical areas, F_1 and F_2, required for each flange, when the maximum allowable unit stresses are assigned, positive for tension and negative for compression. When T, j, d, t, f_1, and f_2, are given:

$$F_1 = -\frac{1}{df_1}\left[Tj + \frac{d^2t}{6}(2f_1 + f_2)\right] \quad \ldots\ldots\ldots\ldots (6)$$

and,

$$F_2 = -\frac{1}{df_2}\left[T(d-j) + \frac{d^2t}{6}(f_1 + 2f_2)\right] \quad \ldots\ldots\ldots (7)$$

Case II.—To find the required area, F_1, for one flange, when the allowable unit stress, f_1, in that flange and the area, F_2, of the other flange, are given,

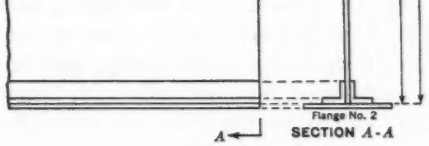

FIG. 17

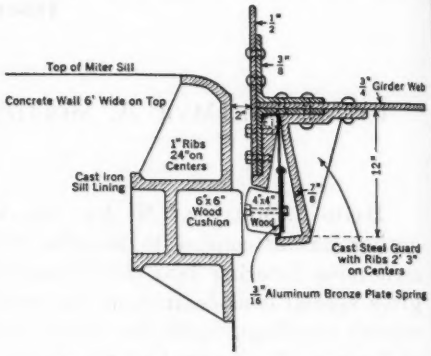
Top of Miter Sill
Concrete Wall 6' Wide on Top
1" Ribs 24" on Centers
Cast Iron Sill Lining
6"x6" Wood Cushion
4"x4" Wood
Girder Web
Cast Steel Guard with Ribs 2' 3" on Centers
Aluminum Bronze Plate Spring

FIG. 18.—MITER-SILL SEALING EDGE

but the unit stress in the given flange is not assigned. When T, j, d, t, f_1, and F_2 are given:

$$F_1 = -\frac{T\left(\frac{6jF_2}{d^2t} + \frac{3j}{d} - 1\right) + \frac{f_1}{2}\left(dt + 4F_2\right)}{2f_1\left(\frac{3F_2}{dt} + 1\right)} \quad \ldots\ldots\ldots (8)$$

and,

$$f_2 = -\left[\frac{6f_1F_1}{dt} + \frac{6Tj}{d^2t} + 2f_1\right] \quad \ldots\ldots\ldots\ldots (9)$$

Case III.—The areas, F_1 and F_2, for the flanges, when the eccentricity of the axial thrust, T, varies between equal limits on both sides of the gravity axis of the section, are given by the formula:

$$F_1 = F_2 = -\frac{1}{6f_1d}\left[3Tj + 2f_1d^2t + \sqrt{(3Tj)^2 + f_1d^2t\,[f_1d^2t - 6T(j-d)]}\right] \quad (10)$$

Equations (6), (7), (8), (9), and (10) look rather lengthy, but will be found very serviceable in designing columns and girders subject to direct stress and bending. The important subject of deflection of lock-gates is not mentioned in this discussion because it has been comprehensively treated elsewhere.[18]

[18] "Kinetic Theory of Engineering Structures," by David A. Molitor, Chapter 14.

Contact at Miter-Sill.—The indeterminate conditions introduced by sill contact can be entirely eliminated in the manner described by the author, but when this is done, there should be provision for forcing centric mitering of the two gate leaves at the instant contact is made on the miter-sill. With only a spring bearing at the sill, the chances for eccentric mitering are very great, and would lead to excessive stresses in the horizontal girders, besides causing delays in operation.

In January, 1912, the writer prepared a design for the lower lock-gate of the Mississippi River Power Company, at Keokuk, Iowa, which embodied several novel features not previously used in mitering gates. Among these was a spring-sill bearing having wooden contact surfaces with guards, and a miter-sill buffer, which forces simultaneous centric bearing of the miter-posts, prior to making spring contacts along the sill. The water pressure,

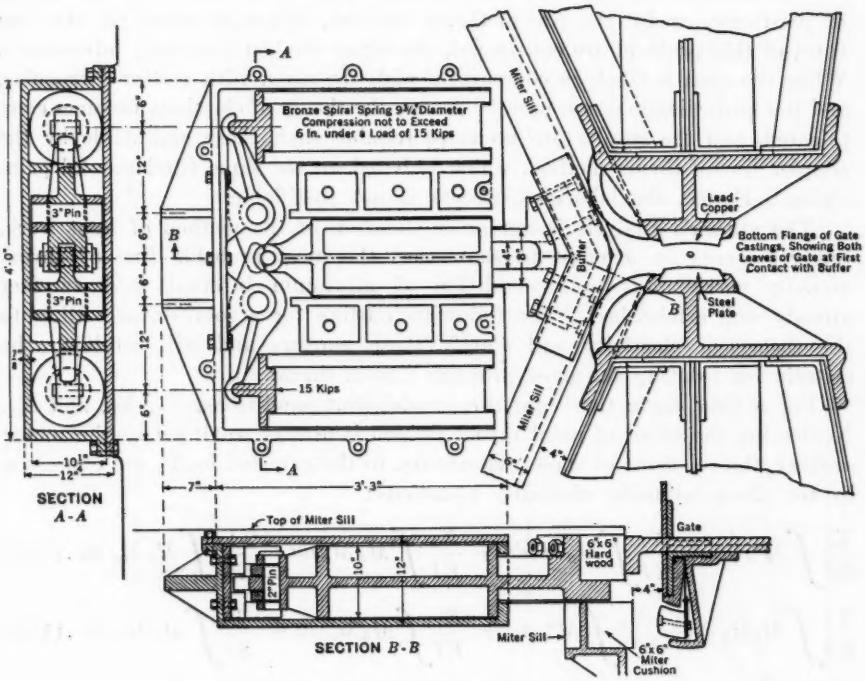

FIG. 19.—MITER-SILL BUFFER

as it gradually increases after closing the gate, forces all contact surfaces along the sill and miter-posts into tight bearing.

The Keokuk gate consisted of two leaves, 49 ft. high and 64.9 ft. wide, for a lock, 110 ft. wide, and a maximum lift of 34 ft. Figs. 18 and 19 show the miter-sill sealing edge and the miter-sill buffer designed for this gate. Hydraulic hand operation was first considered, but preference was given the automatic spring device for the buffer.

Other details may be mentioned only in a general way as the drawings are too extensive for the present discussion. The quoin and miter-post castings were designed to carry renewable metal bearing surfaces, the convex half being of steel and the concave half of lead-copper alloy cast in place, thus ensuring a good fit and saving much machine work.

The anchorage and pintle support were supplied with positive wedge adjustments for "take-up" of wear, and the wearing surfaces consisted of bronze bushings which could be replaced when the "take-up" was eventually exhausted after many years of service.

Side-Wall Analysis.—A side-wall with a large culvert opening, is analyzed as an arch with a superimposed load due to the wall above a certain plane, and acted upon by lateral earth pressure as shown on Fig. 20. All forces in this diagram are in kips (thousands of pounds), for a 1-ft. length of wall.

The solution of the arch portion may proceed according to the theory of elasticity, or by the line-of-thrust method, although when all the features of this problem are considered, the latter method has many advantages. When the arch is thick in comparison with the span, with sections irregular, and the loads diagonal non-parallel forces, the theory of elasticity becomes complicated, and the results are no more reliable than those obtainable by the simpler line-of-thrust method, which is based on the same fundamental principles. Hence, the laborious process is not justifiable.

The author's analysis is not an application of the method of least work, but represents an attempt to show that the most probable line of thrust actually corresponds to a condition of minimum internal work, a fact already well established. The following outline for a solution according to the theory of elasticity will clarify these matters and will establish the criteria for locating the most probable line-of-thrust.

For a fixed arch, there are three redundant conditions, X_a, X_b, and X_c. Neglecting the effect of axial thrust (which is always small for semi-circular arches), the relations of these redundants, to the external loads, are expressed by the three following elasticity equations:

$$\frac{X_a}{EI}\int M^2_a\,du + \frac{X_b}{EI}\int M_a M_b\,du + \frac{X_c}{EI}\int M_a M_c\,du = \frac{1}{EI}\int M_o M_a\,du \quad (11a)$$

$$\frac{X_a}{EI}\int M_b M_a\,du + \frac{X_b}{EI}\int M^2_b\,du + \frac{X_c}{EI}\int M_b M_c\,du + \frac{1}{EI}\int M_o M_b\,du \quad (11b)$$

and,

$$\frac{X_a}{EI}\int M_c M_a\,du + \frac{X_b}{EI}\int M_c M_b\,du + \frac{X_c}{EI}\int M^2_c\,du = \frac{1}{EI}\int M_o M_c\,du \quad (11c)$$

The three redundants, when found from Equations (11), for a given case of simultaneous loads, p, will determine the bending moment for any axial point, m, of the arch, by the following equation:

$$M_m = M_o - M_a X_a - M_b X_b - M_c X_c \quad \ldots\ldots\ldots\ldots (12)$$

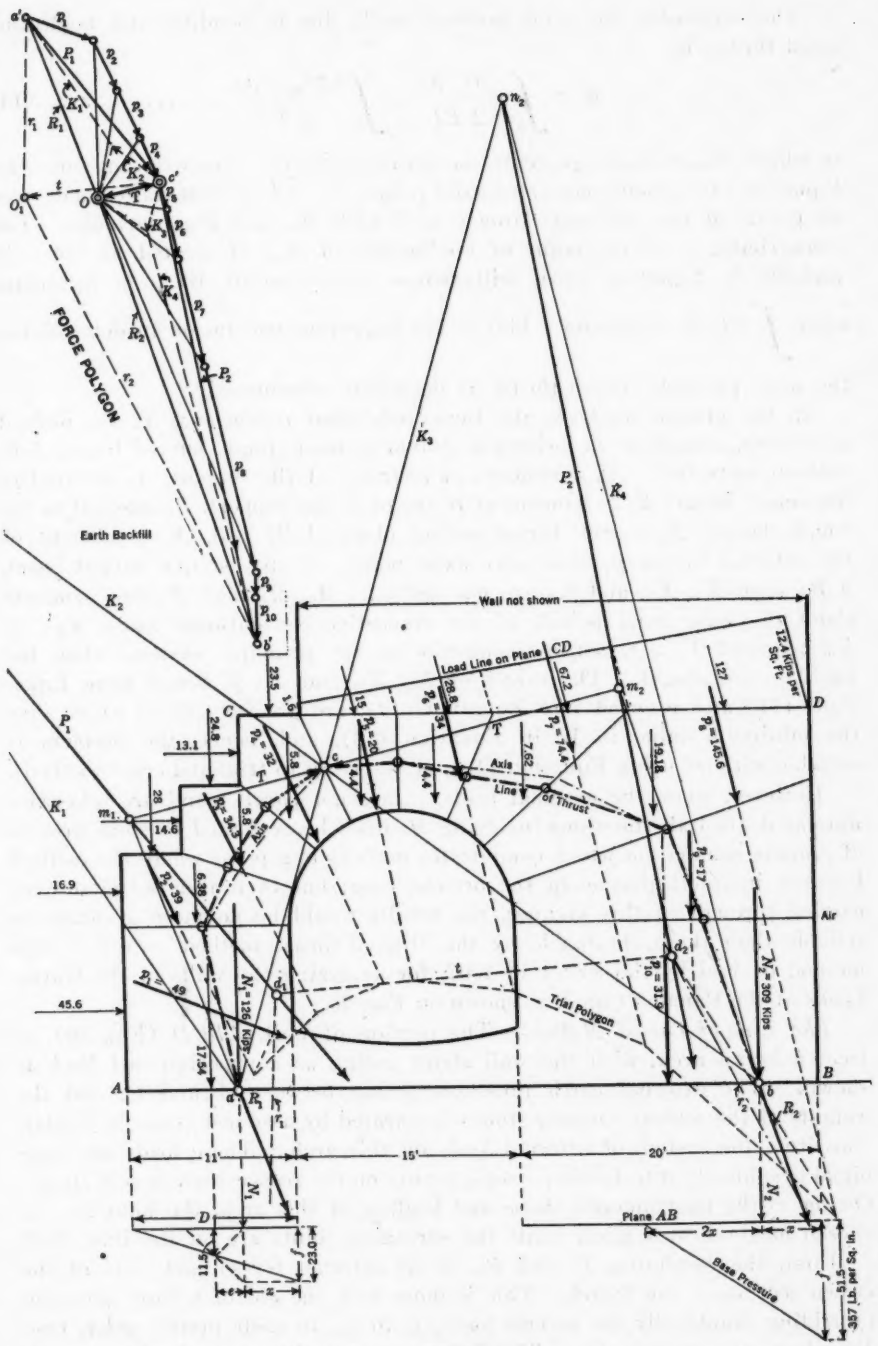
FORCE POLYGON
Earth Backfill
Wall not shown
Load Line on Plane CD
Axis of Thrust
Trial Polygon
Air
Plane AB
Base Pressure
12.4 kips per Sq. Ft.
357 Lb. per Sq. In.
11'
15'
20'

FIG. 20

The expression for total internal work, due to bending and neglecting axial thrust, is,

$$W = \int_0^l \frac{M^2_m \, du}{2\,EI} = \int_0^l \frac{N^2_m \, e^2 \, du}{2\,EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

in which, $M_m = N_m e$ represents the actual moment of the external forces by Equation (12), about successive axial points, m, and the integral extends over all points of the arch axis from 0 to l, with M_m and I as variables. The eccentricity, e, of the point of application of N_m, if treated as the only variable in Equation (13), will reduce the value of W to a minimum when $\int e^2 du$ is minimum. This is the important conclusion in determining the most probable line-of-thrust as described subsequently.

In the present analysis, the three redundant conditions, X, are defined as follows, assuming the principal system to be a simple curved beam, A-B, without restraint: X_a = moment of restraint at the support, A, external to the simple beam; X_b = moment of restraint at the support, B, external to the simple beam; X_c = arch thrust acting along A-B; and, M_o = moment of the external forces, p, about any axial point, m, of a simple curved beam, A-B, when X_a, X_b, and X_c are not acting. M_a, M_b, and M_c are moments about the same axial points, m, for respective conventional loads, $X_a = 1$, $X_b = 1$, and $X_c = 1$, acting separately on the principal system, when the loads, p, are absent. The values of X_a, X_b, and X_c, if found from Equations (11) and inserted into Equation (12), are precisely those which give the minimum value to W in Equation (13), and, hence, the problem is solvable without using Equation (13), or resorting to trial-and-error methods.

However, when the external loads, p, are not parallel and are otherwise unrelated, then all functions involving M_o must be evaluated by some process of graphic summation which complicates matters to a point where the method becomes impracticable as in the present case; but even if a solution were carried through in this manner, the results would be no more accurate or reliable than those obtainable by the "line-of-thrust method" which is the method devised by the writer in 1907, for analyzing the walls of the Gatun Locks of the Panama Canal, as shown on Fig. 20.

The Line-of-Thrust Method.—The portion of wall, *ABCD* (Fig. 20), is treated as an arch, with the wall above acting as a superimposed load as shown. The external earth pressures acting between A and C, and the weights of the several voussoir stones (separated by assumed voussoir joints), constitute the system of external loads on this arch. These loads are combined graphically into Loads p_1 to p_{10}, acting on the respective voussoir stones. Owing to the unsymmetric shape and loading of this arch, the joint at c is chosen as the crown joint, while the springing joints are on the line, A-B.

First, the resultants, P_1 and P_2, of all external forces each side of the crown section, c, are found. This is done with the aid of a force polygon, by adding graphically the several loads, p_1 to p_{10}, in their proper order, true directions, and magnitudes. Then, the resultant, P_1, of the loads, p_1 to p_4,

and the resultant, P_2, of the loads, p_5 to p_{10}, are found in direction and magnitude in the force polygon. A point on the line of action of each is found by drawing an equilibrium polygon, using any convenient pole, O_1, with pole distance, t. The intersections, d_1 and d_2, of the end rays, r_1 and r_2, with the pole ray, t, of the trial polygon, are the desired points, so that P_1 and P_2 are now fully determined. This much of the line drawing including the dotted arch axis, should be drawn in ink so that the next steps, which are first drawn in pencil, may be erased after the final line of thrust is located by trial.

The foregoing really constitutes the preparatory work, and drawing the most probable line-of-thrust completes the analysis, giving the data for stress calculations for all voussoir joints.

The line-of-thrust in an arch ring, is a polygon found by joining, successively, the points of application of the resultant thrusts of the several assumed voussoir joints, as determined by an equilibrium polygon drawn for the given forces. Since any number of equilibrium polygons can be drawn through an arch ring, it becomes necessary to select the most probable one, for which the line-of-thrust satisfies the criterion: That the internal work of deformation, due to the actual loads, p, shall be a minimum. The terms, "equilibrium polygon," and, "resultant polygon," are synonymous, but distinct from the line-of-thrust.

The most probable line-of-thrust, according to the theory of elasticity, is that which reduces to a minimum, the residual departures from the arch center line according to the method of least squares. This is equivalent to making Σe^2 minimum, which is in agreement with Equation (13) for the condition producing the minimum internal work, W.

Hence, if a certain line-of-thrust can be made to coincide with the arch center line, then this particular thrust line will be the most probable one of all the possible lines for the given case of loads, making $\Sigma e^2 = 0$. For any other line-of-thrust, $\Sigma e^2 > 0$. Since values of e may be plus or minus, the sum of the squares can never become zero unless all values of e are zero.

As a first trial, draw a resultant polygon through three points a, b, and c, chosen on the axis of each section. This line, when obtained, will indicate how to shift the three points first assumed, so as to bring the resultant polygon into the closest possible coincidence with the arch axis. After two or three trials the line-of-thrust, $a\ c\ b$, was located as best satisfying the requirement, $\Sigma e^2 = 0_{\text{min.}}$

There is a further condition which must be satisfied as follows: The line of thrust must intersect the arch axis in at least three points, and when the arch and its loading are both symmetrical, there must be at least four such intersections. Fig. 20 shows three intersections as required for unsymmetric cases.

The method of drawing a resultant polygon through three points, a, c, and b, is described as follows: Draw the two lines, K_2 and K_3, through the three given points and continue to the intersections, n_1 and n_2, with the resultants, P_1 and P_2. Then, draw the lines, K_1 and K_4. Now draw four lines, K_1, K_2, K_3, and K_4, in the force polygon, respectively, parallel to the lines just drawn

and complete the parallelogram with the diagonal, $Oc' = T$, thus locating the pole, O, and determining the crown thrust, T, in direction and magnitude. This thrust acts at c, and the line, $m_1 m_2$ parallel to T, determines the final reactions, $m_1 a = R_1$, and $m_2 b = R_2$. The vertical component of R_1 is N_1, and the vertical component of R_2 is N_2, from which the stresses on the sections, AB, are found.

The stresses on any other voussoir joint are obtained from the line of thrust, by completing the resultant polygon for the correct pole, O. The rays, drawn from O to the several corners of the load line of the force polygon, represent the resultants on the several voussoir joints, measured to the scale of forces. The normal components, N, with the eccentricities, e, all scaled from the diagram, determine the stresses on the sections.

The stresses on any voussoir joint are now found from Navier's law for direct stress and bending, as follows:

When $e < \frac{D}{6}$,

$$f = -\frac{N}{D}\left(1 \pm \frac{6e}{D}\right) \quad \text{.....(14)}$$

and, when $e > \frac{D}{6}$, allowing no tension,

$$f = -\frac{2N}{3\left(\frac{D}{2} - e\right)} \quad \text{.....(15)}$$

In Equations (14) and (15), N represents the normal component of the resultant acting on the joint of width, D, and unit thickness, and e is the eccentricity of the point of application of N, as shown for the base sections on A-B (Fig. 20).

It is easily seen from Equation (14) that when the line-of-thrust remains everywhere within the middle-third, there will be no tensile stresses.

This completes the analysis, on a theoretically sound basis and with sufficient accuracy for all practical purposes. Had the author applied the criteria as here presented, for identifying the most probable one of all possible lines-of-thrust, all his involved calculations would have been avoided.

In Table 1 the sum of Column (7), representing the internal work due to axial thrust, and treated as a constant, contributes nothing of value, for which reason the author should have neglected axial thrust. If the sums are taken for Columns (9), (12), (15), (18), and (21), it will be found that Column (18) gives $\Sigma e^2 =$ minimum, which is the only conclusion of any value. These sums are not a true measure for minimum Σe^2, because the voussoirs were not taken at constant distances apart on the axis; hence, dl should have been taken in as a variable factor.

The author further states, following Equation (5):

"There are three unknown quantities to be obtained, the horizontal and vertical components, and the point of application of the crown thrust, and all are determined very easily by this process."

If he means by this that these three unknowns constitute the three redundant conditions involved in the arch problem, then he is seriously in error, because the three unknowns of which he speaks, merely determine one of the three redundants. He has thus solved a threefold statically indeterminate problem by considering only one.

All methods for the evaluation of the three redundant conditions in arches of the kind here treated, are more or less speculative; but they yield results which are within knowable limits, especially when the heterogeneous character of building materials is considered. Whether the resultant polygon and line-of-thrust are located correctly by the method of least work, by the elastic theory, or by a few trials observing the criteria stated herein, makes very little difference except in the labor involved, and certainly the least laborious solution is the most acceptable. If the problem dealt with large flat arch ribs, having thin rings and carrying vertical loads, then the theory of elasticity would be applicable and would afford the best and most accurate method of arch analysis.[19]

CHARLES A. D. YOUNG,[20] M. AM. SOC. C. E. (by letter).[20a]—The author has presented a thorough description of the many interesting problems which arise in the design of navigable waterways and the accompanying locks and dams. The adaptation of the principles of the Venturi meter to the design of the culvert-valve chambers is a valuable contribution to the art, and the results will be watched with interest by all concerned with the design of locks. If the results obtained in operation are as anticipated, this design, no doubt, will be adopted wherever applicable because it will afford a material saving in the cost of the valves and their operating machinery. It is to be hoped that tests of this installation will be made and the results published for the benefit of the profession.

It is only within comparatively recent years that the scientific design of lock-gates has been given much consideration, and as yet no method has been devised for determining the stress transferred to the upper girders by the vertical framing when the gate is in contact with the sill. It has been the custom to compensate for these indeterminate stresses by adding to the water load a certain amount of static head at the upper part of the gate; but by eliminating the cause of the indeterminate stresses as the author does, the design of a lock-gate becomes a comparatively simple problem and this excess head may be omitted.

In the design of the gates, as described by the author, there is no contact at the sill and hence no indeterminate stress; but it is noted that an allowance of a 15-ft. head at the upper part of the gate was made to compensate for shock.

Standard specifications for structural steel construction provide that no steel less than $\frac{5}{16}$ in. thick shall be used. Due to the fact that lock-gates are particularly susceptible to corrosion, it is considered good practice to limit

[19] "Kinetic Theory of Engineering Structures," by David A. Molitor, Chapter 15.

[20] Senior Engr., U. S. Reclamation Bureau, Denver, Colo.

[20a] Received by the Secretary January 11, 1932.

this thickness to $\frac{3}{8}$ in. When the girders are equally spaced and of equal depth, it is found that the upper ones will be much stronger than required to resist the static head.

No doubt the spring seal, described by the author and designed to eliminate pressure between gate and sill, will provide an efficient seal, but experience with similar seals on the Stoney culvert valves at Panama, and at the Lake Washington Canal Lock, at Seattle, Wash., indicates that it may be short-lived. In the cases cited, after operation for a short time, the spring broke near the point of attachment to the valve, presumably due to vibration caused by slight leakage. The springs were replaced by heavy rubber belting and no further difficulty was experienced.

The first attempt to eliminate pressure at the sill that has come to the writer's attention, was in the design of the lock-gates for the double-lift lock

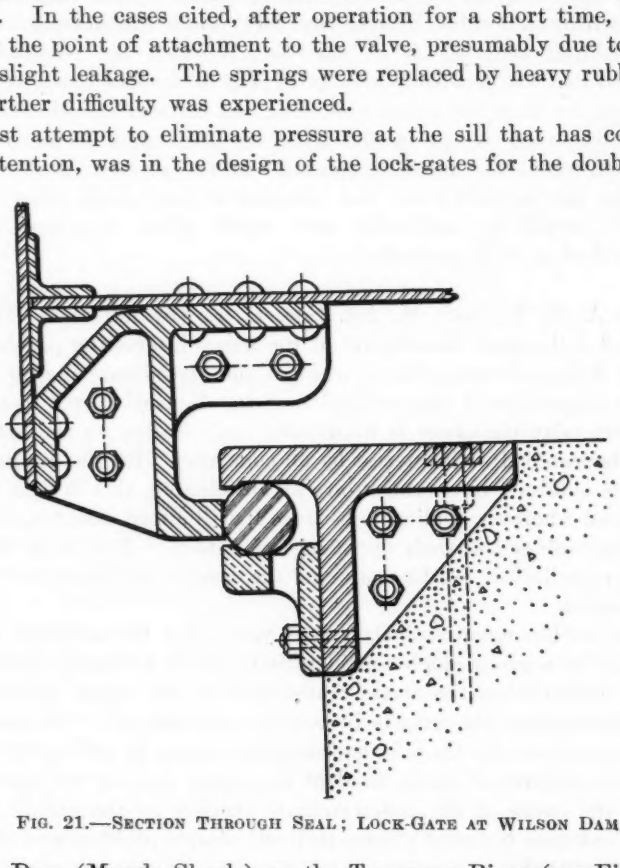

FIG. 21.—SECTION THROUGH SEAL; LOCK-GATE AT WILSON DAM

at Wilson Dam (Muscle Shoals) on the Tennessee River (see Fig. 21). As the head is increased behind the gate, the water pressure acts through the holes in the lower support and forces the solid rubber bar against the gate. This same method was used by the writer on the gates for Lock and Dam No. 1, Tennessee River, at Florence, Ala., and in both cases was very successful in operation.

Adjustable bearing-plates backed by Babbitt metal, similar to those described by the author, were used on the lock-gates of the Lake Washington Canal. In this case the gates were swung to the closed position, the plates were forced out by set screws until they were in perfect contact, and the

molten metal was poured in from the top. In order to insure that the space was completely filled with the Babbitt metal, the plates were removed, and large cavities were found due to the molten metal congealing on the sides near the top and clogging the opening. This difficulty was eliminated by drilling holes about 4 ft. apart and pouring in 4-ft. lifts.

It is interesting to note the effect on the stresses, of placing the sheathing plates on the down-stream side of the gate. Such placing of skin-plate has been considered an advantage in single-skin gates in order to reduce the uplift on the bottom when the gate was closed; but as this puts tension in the rivets that hold the skin-plate, it has seldom been done. One case only has come to the writer's attention and that was in the Tennessee River lock, at Riverton, Ala. In this case, the girders were I-beams, and it is doubtful whether the designer had any other object in mind than the reduction of uplift. From an operation standpoint such an arrangement is objectionable because in a silt-bearing stream, such as the Tennessee, the girders fill with mud and débris which is difficult to clean out for painting.

The writer agrees with the author that gate anchorages made of eye-bars are difficult to set, but it is his experience that those built of structural members in the shape of a triangle are easily set and held in position, since they are rigid and may be held in place by anchor-bolts. This type of anchorage was used on the Lake Washington Canal Lock. The type described by the author has the advantage perhaps that it does not extend deeply into the concrete and can be placed after the concrete has been poured.

The method of adjusting the rotation axis of the gate as described by the author, was used on Davis Lock at Sault Ste. Marie, Mich., in order to secure the fine adjustment required and, at the same time, to provide the necessary strength of threads, and it has been used on other locks. Observation of the difficulties encountered in removing a gate-leaf of the Poe Lock at Sault Ste. Marie, Mich., prompted the writer to introduce the two eye-bars which can be swung out of the way and permit the gate to be lifted off its pintle without changing the adjustment.

Since the change in temperature of a lock-gate may materially change its location when mitered, a spring has been introduced in the spar, in some cases where the bull-wheel is used, to allow full stroke for all conditions. This spring also acts as a cushion to relieve the machinery of undue strain in starting and stopping the gate.

The addition of air chambers to lock-gates is of questionable value since the anchorages and pintle must be made of sufficient size to carry the load of the gate when swinging in air, which is necessary during construction for the purpose of making adjustments and, at other times, when the lock is unwatered for painting or repairs. It is true that the amount of swinging done under these conditions is small compared to the operation conditions, but if the pintle and gudgeon pin are conservatively designed to carry the full load of the gate in air, the wear on these parts will be kept at a minimum.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

WESTERN HIGHWAY PRACTICE, WITH SPECIAL REFERENCE TO CALIFORNIA

Discussion

BY MESSRS. H. S. KERR, E. Q. SULLIVAN, AND T. A. CORRY

H. S. KERR,[5] M. AM. SOC. C. E. (by letter).[5a]—While reading the part of this paper relative to protection from the elements, the writer was reminded of the success achieved by the Utah Highway Department in keeping all important highways open throughout the four winters from 1927–28 to 1930–31. As a matter of fact, the snowfall was below the average during these years, but even had the average been exceeded the organization could successfully have maintained continuity of traffic on all Utah interstate highways with perhaps short minor delays due to high winds. On one of the through highways, elevations range between 6 500 and 8 000 ft. for a distance of 70 miles. The snowfall is heavy and drifting is of frequent occurrence. The design of the highway was for a grade line well above the valley level, wherever possible, giving the wind a chance to help remove the snow.

Over portions of this section, when it was not possible to raise the grade, and in the few cases where cuts were necessary, snow fencing has proved advantageous. Fencing of this kind is used in numerous other parts of the State, Fig. 16 being a view of a section of the Bingham Highway, 15 miles southwest of Salt Lake City. This is the common slat fence consisting of laths about 2 in. apart, held together with wire. The posts are old railroad boiler tubes cut to the desired length. The fence is not wired tightly to posts, thus enabling it to be raised as drifts form and to be removed in the spring.

Protection from Floods.—Fig. 17 is a view of the crossing of "West Water" Wash, in Grand County, near the Colorado-Utah State line. This wash is typical of those encountered on the desert of Eastern Utah, across which U. S. Highway No. 50 is located for a distance of 130 miles. Aside

NOTE.—The paper by C. S. Pope, M. Am. Soc. C. E., was presented at the meeting of the Highway Division, Sacramento, Calif., April 24, 1930, and published in November, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1932, by Messrs. J. S. Bright and R. D. Rader.

[5] Chf. Engr., State Road Comm., Salt Lake Ctiy, Utah.

[5a] Received by the Secretary December 15, 1931.

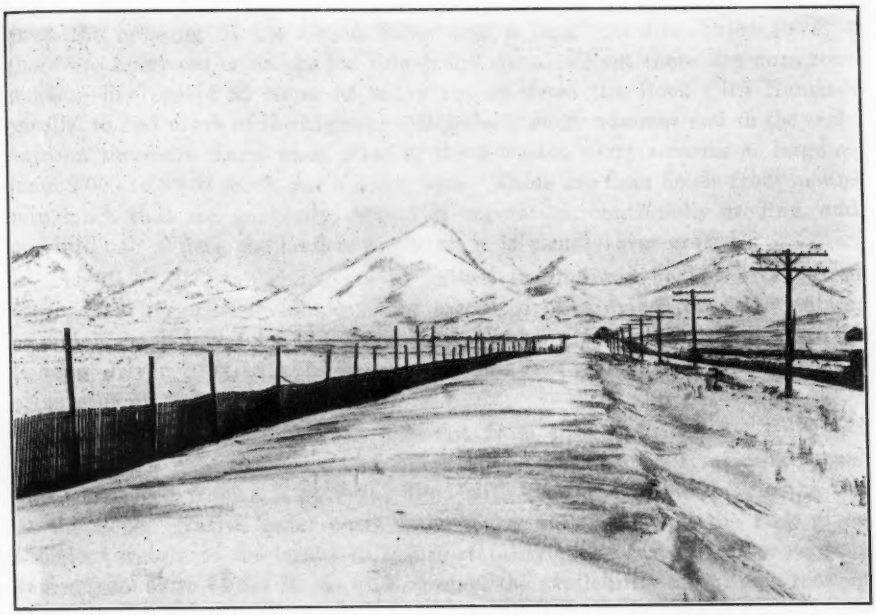

FIG. 16.—FORMATION OF DRIFTS AT SNOW FENCE ALONG BINGHAM HIGHWAY, IN UTAH.

FIG. 17.—BRIDGE OVER COTTONWOOD WASH, IN UTAH.

from the crossing of the Green River and a small stream—Price River—there are no rivers or creeks for this entire distance; yet there are numerous washes—dry except at times of heavy run-off from the Book Cliff Range—parallel to and north of the highway. Regularly every summer and in the early autumn there are times when some of these washes carry streams as large as from 2 000 to 3 000 sec-ft. for a short time. These are flash floods from mountain lands that are, generally, devoid of vegetation, continually eroding, and precipitous. Where the land is not bare, it is usually over-grazed.

The State of Utah endeavors to protect its highway bridges from these erroneously termed "cloudbursts," by creosoted piles driven on 5-ft. centers, to which are bolted 4 by 12-in. creosoted planks. Normally, the piles are 25 ft. long, driven 20 ft. into the stream bed. The cost varies from $5 to $8 per ft. of revetment.

A form of basket somewhat different from that described by Mr. Pope has also been used to good advantage. These baskets consist of cedar posts and wire mesh frames, 6 by 6 ft., filled with rock and located along the toe of the slope. Native cedar posts resist decay very well, and the rock gives sufficient weight to the basket to insure stability. The cost of this protection varies from $2 to $4 per ft., depending upon the availability of suitable rock.

Experience in Utah has demonstrated the fact that wooden bridges are more satisfactory for the desert section of Eastern Utah, except for the 15-mile section just east of the City of Price, where traffic is much heavier. Here where the wash channels are deeper and more permanently located, standard steel or concrete bridges are being built, in connection with the reconstruction of the highway. Farther east, however, there is danger of losing a permanent structure as easily as one of wood, if an extra heavy run-off occurred at one of the fifteen major washes bridged on this section between the Colorado State line and a point 15 miles east of Price.

In addition to the Eastern Utah desert there is another desert in Western Utah crossed jointly by U. S. Routes 40 and 50. This desert west of Great Salt Lake is itself a desiccated lake bed and presents construction and maintenance problems different from those of the Eastern Utah desert. While generally dry, there are times when an extensive shallow salt lake is formed. Protection against wave action by means of gravel beaching was required for a 10-mile section. No major drainage structures were necessary, but minor openings were required at frequent intervals. These openings are in the form of small wooden bridges constructed of untreated timber. The water is extremely salty (25%) and is a natural preservative for these wooden structures. The use of concrete and steel could not be considered. The wooden structures were built by the use of wooden pegs in lieu of spikes.

Aside from those noted herein, there are no special problems in this State; however, on account of the topographic conditions prevailing, location problems present many interesting and rather unique features.

E. Q. SULLIVAN,[6] Assoc. M. Am. Soc. C. E. (by letter).[6a]—Little or no rain falls in the Western States during certain seasons of the year. In California, the summer is practically without rainfall. Dry grass on the roadsides offers an extreme fire hazard due to motorists throwing out matches or cigar butts. Special thought has been given to the design and maintenance of roads to reduce this hazard. In the original construction of roads through areas where such fires are likely to occur, the right of way is cleared for a distance of about 10 ft. back from the edge of the pavement. This is followed by maintenance operations, consisting of clearing away all dry grass and brush in advance of the dry season.

Effect of Water Rights on Highway Design.—In the semi-arid West water rights may become a controlling factor in highway design. For example, in the recreational mountain regions of Southern California a spring of water used for summer cabins is of such great value that its location may necessitate a considerable expenditure if it is found in the path of highway location.

Much of the irrigation water used in the valleys of California and in some other Western States is pumped out of the gravel deposits formed by the discharge of streams on to valley floors. Such streams may be dry for most of the year. It is customary for water users to promote the absorption of flood waters in these natural gravel deposits which then act as reservoirs. Water is pumped out of the gravel deposits for irrigation during the dry season. In order to conserve the greatest possible quantity of water, the stream may be guided in meandering over the gravel deposits by the use of light check dams or artificial channels. Works of this kind are constructed by water users. The success of absorption is dependent on minimizing the percentage of silt carried in flood waters. Silt tends to seal up the gravel deposits and thus prevents the absorption of such waters.

The location of highways through drainage areas above such gravel deposits is subject to attack by water users on several grounds. It is sometimes claimed that the opening of the drainage area with the highway increases fire hazard; in the event, the drainage area is burned over, the soil and ashes brought down seal up the gravel deposits and prevent absorption of flood waters.

Claims are also made by water users that earth and rock dust exposed to the weather in highway cuts or fills are washed down the streams to seal up gravel deposits, thus preventing the absorption of flood waters.

In order to obviate damage claims, it is necessary to build check dams across streams to retain silt originating from highway construction. The objections raised to increased fire hazard are overcome by clearing and by maintaining the cleared strip as described.

Control of Snow on Recreational Roads.—The use of recreational roads during and following heavy snowfall has become increasingly popular. This has affected design and maintenance.

The problem of caring for the public is very acute during snow storms on some of the recreational roads. The cities of California have no snow-

[6] Dist. Engr., State Div. of Highways, Dist. VIII, San Bernardino, Calif.

[6a] Received by the Secretary January 4, 1932.

fall and the public, in general, is poorly equipped to enter mountain regions in the snow. Because of the novelty, crowds rush to the mountains whenever snow begins to fall and the roads, therefore, must be kept open continuously.

The problem is solved by giving special attention to the location and construction of recreational roads, with the thought in mind that their major use by the public may be during snow storms. Large parking places are provided at elevations below the snow line where cars can be stopped to put on chains. The highway is located so as to avoid snow slides and it is also built, as far as possible, on southern exposures. Drifting snow is taken care of by snow fences and other devices.

The more popular recreational roads are kept open during snow storms by the continuous use of snow plows, and traffic is aided and controlled to allow the maximum use of the roads by the crowds of pleasure seekers.

T. A. CORRY,[7] M. AM. SOC. C. E. (by letter)[7a]—Referring to the part of this paper headed "Sand Dunes," the writer would like to call attention to similar problems encountered on the railway that crosses La Joya Desert in Southern Peru. The winds are almost constantly in the same direction and rarely exceed more than 20 miles per hour.

The sand dunes are all shaped in the form of crescents and they move from 40 to 50 ft. annually, depending on their relative sizes. Their derivation is from the decomposition of granitic rocks.

In Southern Peru the practice is to control this movement by burying the dunes under a thin layer of gravel. Section gangs can cover a dune large enough to be included in a circumscribed square, say 100 by 100 ft., at a cost of about $7.00.

To adapt the same process to California conditions, where the winds are higher, would probably require the use of very coarse gravel.

[7] Chf. Engr., Southern Rys. of Peru, Arequipa, Peru.
[7a] Received by the Secretary February 23, 1932.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

LOW-COST BITUMINOUS ROADS

Discussion

BY MESSRS. A. H. HINKLE, CLAUDE P. OWENS, CARLTON N. CONNER, HAROLD L. TILTON, AND L. I. HEWES

A. H. HINKLE,[12] M. AM. SOC. C. E. (by letter).[12a]—Bituminous types of surface, such as those described by Mr. Frickstad, require, as a background, the use of a binder that hardens slowly so that a smooth surface can be developed during the hardening process.

The problem, so far as it affects the bituminous binder, is to select one that can be readily incorporated in the aggregate and that will give satisfactory results in holding the aggregate in place after it is incorporated. The lighter or more liquid the oils, asphalts, and tars, the better they will penetrate the fine aggregate. However, the heavier grades are more durable if they can be made to "stick" to the surface of the aggregate, which is sometimes dirty. Thus, attention is at once brought to the use and study of the emulsified and cut-back asphalts, or similar acting bituminous materials, which furnish both qualities in that they are temporarily liquid with high penetration properties, and after the evaporation of the water or distillate there remains the original heavy, asphaltic product that gives the necessary bond and wear. It so happens that the fuel oil of California (and some other fields as well) are by nature, in such a liquid state that they need little preparation before their use in the work.

It is apparent that climatic and local conditions may affect the details of construction somewhat, as well as the results secured. A more thorough understanding of the proper use of light, medium, and heavy grades of bituminous products in bituminous mixtures and bituminous surface treatments will aid appreciably in the better results secured in developing low-cost roads.

NOTE.—The paper by Walter N. Frickstad, M. Am. Soc. C. E., was presented at the meeting of the Highway Division, Sacramento, Calif., April 24, 1930, and published in November, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: February, 1932, by Messrs. H. J. Spelman, J. T. L. McNew, Roger M. Lee, E. Q. Sullivan, and Hugh W. Skidmore.

[12] Supt. of Maintenance, Indiana State Highway Comm., Indianapolis, Ind.

[12a] Received by the Secretary December 21, 1931.

Surface Treatment.—A light cut-back asphalt, an emulsified asphalt, or tar of about the same viscosity, might be substituted for the light fuel oil as a first surface treatment of fine-crushed aggregate roads mentioned by Mr. Frickstad. After the first, or perhaps the second, treatment (that is, after a crust has been formed), the use of a heavier bituminous product will generally produce a more economical treatment. This heavier asphalt, or tar, might be classified for this discussion into "medium" and "heavy."

The medium grade has a viscosity such that it will permit dragging the treatment after the covering aggregate is applied and, in the dragging operation, the aggregate coated with the tar or asphalt will be deposited in the low places on the surface, thus greatly smoothing it. This medium grade of bitumen may vary in viscosity for cut-back asphalt from 50 to 200 (Engler, at 50° cent.); for tar from 30 to 80 (Engler, at 40° cent.). The lighter viscosities should be used in cool weather. The heaviest viscosity that can be used satisfactorily at the available air temperatures will produce the most economical work and will be most satisfactory from the point of view of early drying. The heavy grade of bituminous material, in addition to being a heavy fuel oil, as mentioned in Mr. Frickstad's paper, might be an asphalt with a penetration of 175 to 250, at 25° cent., or a tar having a float test of 120 to 180, at 32° cent.

If the heavy grade of tar or asphalt is used, it should be covered immediately with $\frac{3}{4}$-in. to 1-in. maximum size stone, slag, or gravel (80 to 110 lb. per gal.), and may then be immediately "traveled" without the inconvenience of splashing on machines. This heavy bituminous material, however, does not have the advantage possessed by the medium or lighter grades of permitting the dragging of the surface after treatment.

Most bituminous pavements are subject to slight distortions and wear under traffic which gradually roughens their surfaces. These slight irregularities frequently can be eliminated by proper dragging with a long base drag just after surface treatments with a medium grade of tar or asphalt; such treatments are applied periodically as regular maintenance operations.

Mixing Treatment.—The "mixing treatment" (sometimes called "mulch treatment"), involving a light cut-back asphalt or light tar (specific viscosity, Engler, 40° cent., of 10 to 20), has been used extensively in some of the Central and Eastern States. Although light and medium grades of tars and asphalt oils differing from the fuel oils used in California have been generally used in this work, and although some of the details of the mixing operations have differed, the general process has resulted in producing an intimate mixture of the bitumen and loose aggregate, which is leveled and spread over the surface and compacted by traffic, sometimes aided by a roller. In this work, California fuel and similar oils in climates subject to heavy moisture combined with deep frost action, have been used enough to indicate their possibilities. It is yet to be fully demonstrated that they will form a surface as resistant to traffic and other destructive agents as a tar or cut-back asphalt made from an asphalt base having a penetration of 70 to 110.

It has been observed that where a layer of comparatively coarse aggregate was treated on the road by the "mixing" process, a much more stable crust was

produced. This led to the development of what is termed, for want of a better designation, a "bituminous retread top" of 2 to 3-in. compacted thickness of stone about 2 in. in size. This differs primarily from the oil-mixing method described by Mr. Frickstad in that a coarser aggregate is used and the bitumen consists of cut-back asphalt, an emulsified asphalt, or a medium grade of tar. While many variations in the details of construction have occurred in the past, this work has now been more or less standardized in Indiana, as follows.

Any time from one week to six weeks after the oil is applied to the old stone or gravel surface, crushed stone, slag, or gravel (preferably ranging in size from $1\frac{1}{2}$ to $2\frac{1}{2}$ in.), is spread on the surface from trucks to a depth of about $3\frac{1}{2}$ in. This aggregate is leveled by a heavy grader and the edges are "trued up" by hand. To this loose aggregate is applied about 0.5 gal. per sq. yd. of the medium tar, cut-back asphalt, or emulsified asphalt. The aggregate is then harrowed with a spike-tooth harrow, struck off with a grader, and rolled once immediately thereafter; as soon as the bituminous material hardens sufficiently so that the aggregate will compact and remain compacted, it is again rolled. A second application of bituminous material of about 0.4 gal. is applied and, after curing, the surface is again thoroughly rolled. Just preceding the third application of bituminous material, the voids in the coarse aggregate are filled with $\frac{3}{4}$-in. to $\frac{3}{8}$-in size aggregate leaving a uniform and slightly excess layer of covering on the surface. Fig. 5 shows the arrangement of a brush and an old carpet, pulled by a small truck, used for this purpose. This covering is not rolled until after the third application of about 0.35 gal. of bituminous material, after which the surface is planed or dragged and again thoroughly rolled. Equipment for dragging this covering material after the third application is shown in Fig. 6. This process rolls the coated stone chips around over the surface until they drop into the voids or fill the depressions in the surface. (This same process of dragging is used, after re-treatment, in future years, with a medium grade of bitumen to restore the smoothness of a surface that may have been roughened under traffic.) A fourth treatment of about 0.17 gal. per sq. yd. of the bituminous material covered by $\frac{1}{2}$-in. size aggregate may be used if necessary. This process resembles the "mixing treatment" in its first stages (except for size of aggregate) and its finishing operations are like the bituminous (penetration) macadam method.

The work of smoothing and leveling is largely done mechanically, and experience has shown that generally a smoother surface is produced and at a much less cost than with an ordinary penetration macadam. In Indiana, the cost of this top course, 3 in., compacted, including the oiling of the old road, for a 20-ft. width, has been from $3 800 to $6 000 per mile. Experience has shown that where this top is built on a suitable base, it will carry any ordinary traffic that may be found on the average State highway. The surface is maintained the same way as the ordinary bituminous macadam. In this bituminous retread top a low percentage of bitumen is used, which is made possible by means of the coarse aggregate.

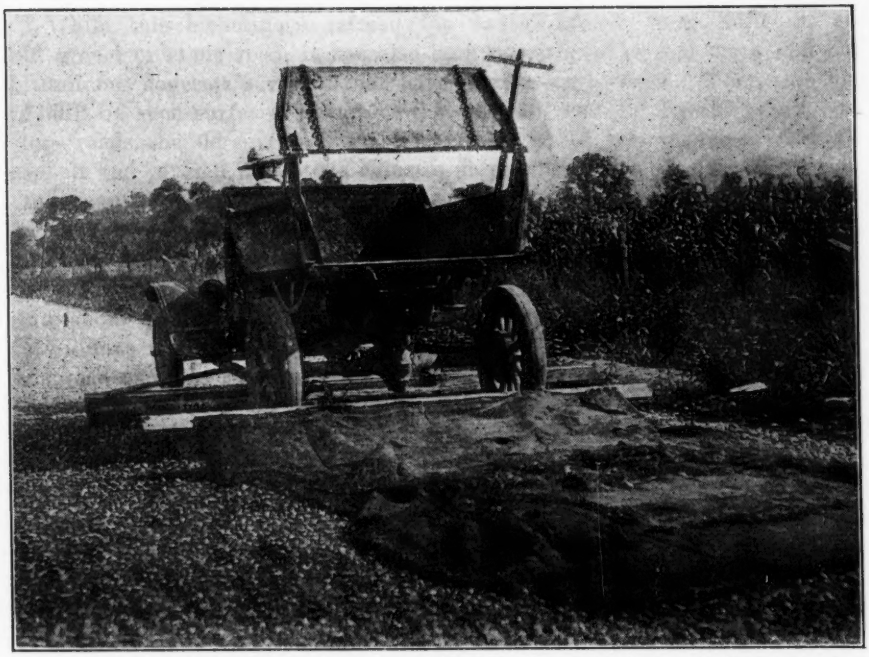

FIG. 5.—EQUIPMENT FOR BRUSHING THE COVERING MATERIAL INTO THE VOIDS OF THE STONE.

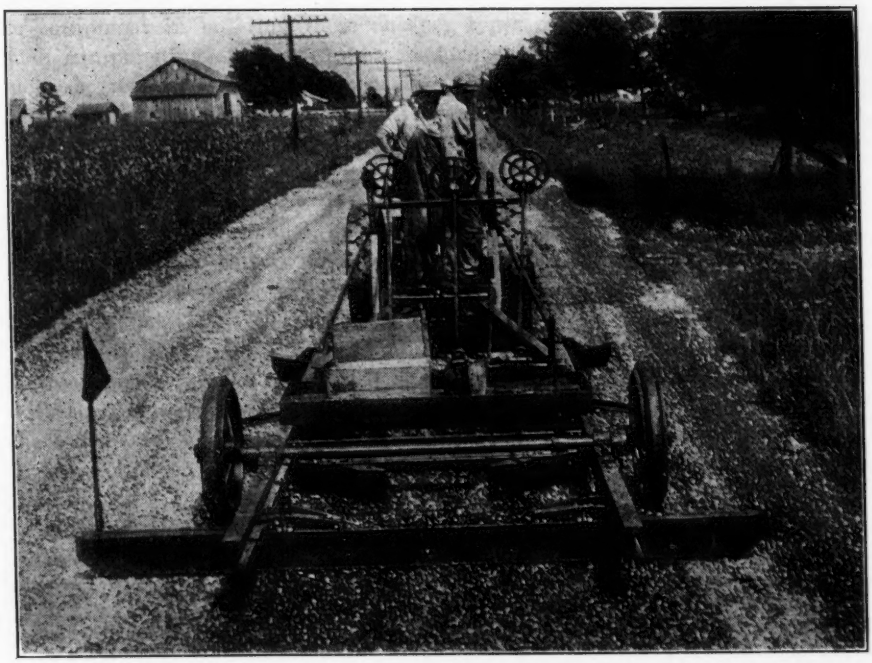

FIG. 6.—DRAGGING THE COVERING AFTER THE THIRD APPLICAION OF BITUMINOUS MATERIAL.

While this bituminous retread top has ordinarily been built on an old gravel or stone road, it has also been constructed on old brick and old bituminous concrete surfaces that have become very rough. When this top is built on such surfaces, in lieu of the first oil treatment for the gravel and stone roads, the old surface is given about 0.2 gal. of medium tar or cut-back asphalt and covered with stone chips to prevent the layer of coarse aggregate (to be applied later), from sliding and shoving on the old smooth surface while construction operations are under way.

Plant Mix.—As an alternative to this method of mixing coarse aggregate on the surface, a plant-mix method is being advocated, by which aggregate about $2\frac{1}{2}$ to $1\frac{1}{2}$ in. in size, unheated, is coated with a medium grade of bituminous material before it is hauled to the road. This coated aggregate is spread upon the surface with the road grader and other machinery in much the same manner as the aggregate is spread in the road-mixing process.

This coated aggregate is compacted by rolling, after which a thin spread of $\frac{3}{4}$-in. coated aggregate is spread on the surface and the surface is again rolled. A system is now in use by which this aggregate is coated by immersing it in the asphalt or tar as it is unloaded by conveyors from the car. These methods of making a plant mix of coarse aggregate bear about the same relation to the bituminous retread described in this discussion, as the plant oil-mixing method described by Mr. Frickstad bears to the road mix with the same grade of fine aggregate.

Equipment.—Perhaps the use of a long-base heavy drag or planer described under "Surface Treatment" in this discussion, is the greatest development in equipment in building these cheaper types of bituminous roads. With such equipment a surface can be made even smoother than by any hand methods yet devised. This smoothing applies especially to the oil-treated base for the retreaded top where it is necessary to start producing a smooth surface if a smooth-riding, finished, top course is to be obtained.

Generally speaking, the equipment for this type of work may be that used in maintaining an ordinary traffic-bound gravel or stone road. While special machinery may be provided, as Mr. Frickstad states they are doing in some places, fortunately this is not necessary for counties and other smaller governmental units which will have occasion to build short stretches of this type of surface.

Grading of Aggregate.—The method described herein under "Bituminous Retread" calls for the use of a comparatively coarse aggregate as compared with the fine aggregate used in methods described by Mr. Frickstad. With the coarser aggregate the question seems to be largely one of sufficient base in order to carry the loads rather than a question of sufficient durability of the top.

This coarse aggregate method has been used on some of the most heavily traveled roads in Indiana, and it can be said that, except for the necessity of treatments as a maintenance operation every year or so (which is some inconvenience to the traffic), the surface is holding satisfactorily.

Bituminous Materials.—It is fully demonstrated that cut-back asphalts, emulsified asphalts, and tars, are perfectly satisfactory for this work. When

an asphalt product is used, the writer is inclined to think that best results will be secured by the use of an asphalt having a base with a penetration of about 80 to 120 and that it matters little whether this asphalt is put in a liquid form as a cut-back product, or as a suitable emulsified product.

While the fuel oil of California, and other oils, may not exactly comply with these requirements, they do not seriously depart therefrom; also the lower cost of the fuel oil and similar oils may more than justify their use even if they are not quite so satisfactory under certain conditions. With the use of fine aggregate the first coat of bitumen at least must be a very light tar, fuel oil, emulsified asphalt, or cut-back asphalt with a suitable distillate, as other grades will not readily penetrate such aggregate.

If further experiments with fuel oil mixed with aggregate containing an excess of fines proves successful in all climates, it will have great merit in those areas where much local fine aggregate is available, since, frequently, the local pits will supply this aggregate in about the proper proportions of fines that Mr. Frickstad describes as being used in California.

With the use of a coarse aggregate the percentage of bitumen is greatly reduced. For instance, the quantity used to coat the $1\frac{1}{2}$ to $2\frac{1}{2}$-in. stone will range from 2 to 3 per cent. With this coarser aggregate method there must, of course, be some finer aggregate used to seal the top and this requires a greater percentage of bitumen. Generally, however, the total in the coarse aggregate type, including the seal, is less than $4\frac{1}{2}$ per cent. The low percentage of bitumen contributes to a lower cost and the coarse aggregate, if it makes any difference at all, makes a crust capable of carrying heavier traffic than the fine aggregate.

Due to the heavy base required in a country subject to deep frost action and moisture, it is doubtful whether new construction of this type will make a rapid advancement in such climates. If started from the ground to build up a base, the cost of this type would approach closely that of one of the standard pavements now (1932) being constructed. This is due to the extremely heavy base required on clay soils where the frost action and moisture are pronounced. However, in such climates, there are usually many miles of old stone and gravel roads which have been used for years and which frequently have a well compacted and solid base. These roads (especially where heavy trucking is not likely to develop), are very suitable foundations on which to build this low-cost bituminous surface. Frequently, they will serve the public as well as a higher type of surface.

Standardization of Types.—While a great variety of bituminous mixture is used as a top course (consisting of various sizes of aggregates and numerous kinds and grades of asphaltic oils, cut-back asphalts, and tars), it appears to the writer that the tendency is toward standardization into certain specific types which may be classified under the following five general classes, the first three of which are described by Mr. Frickstad:

1.—Surface treatments on traffic-bound gravel and stone roads wherein a light grade of bituminous material is used as a first application on the clean, compacted surface consisting largely of a fine aggregate; and, then, by suc-

cessive treatments with medium grades of bituminous materials and the addition of aggregate covering, a bituminous crust is built up sufficiently strong to carry the traffic.

2.—The bituminous "mulch" or "oil mix" method wherein the light grade of bituminous material is mixed with fine aggregate on the surface to a depth of from 1 in. to 4 in.

3.—Coating the road with a plant mix 1 in. to 3 in. deep, wherein the mixture consists of an aggregate containing a large percentage of fines, which is spread on the road from a truck and leveled with a grader and other road machinery and maintained by blading and scraping until it hardens and traffic compacts it. (This mixture is virtually that described under Class 2, except that it is a plant mix instead of mixed in place.)

4.—A bituminous retread top made by mixing on the road an aggregate ranging in size from $2\frac{1}{2}$ in. to 1 in. by successive applications of a cut-back asphalt, emulsified asphalt, or medium grade of tar, the first stages of this method resembling Class 2, and the final stages resembling the penetration macadam type.

5.—A plant mix of the same size of aggregate (unheated) as that used in Class 4, and an emulsified asphalt, a cut-back asphalt, or medium grade of tar made either by mixing the material by ordinary mixing-plant methods or by immersing it in such bituminous materials, the coated material being hauled to the road and spread in the same manner as in Class 3, and which, after thoroughly rolling and curing, is coated with $\frac{3}{4}$-in., coated aggregated applied and spread in the same manner, using a comparatively lean mix and sealing the surface with an ordinary seal or surface treatment coat if necessary.

In all the foregoing methods the low cost of the finished surface is brought about by the fact that the aggregate is handled, spread, and finished mostly by mechanical means, thus reducing the hand labor and expensive costs to a minimum; also, from the fact that cheaper processes are used in the mixing methods made possible by not heating the aggregate and in using a bitumen with a low viscosity and because frequently a less costly aggregate can be used in some of the work. Inasmuch as the aggregate is unheated in all these cheaper methods, it goes without saying that they are most suitable for use in a dry, warm climate, or during the dry, warm season in other climates.

Base Required Varies with Climatic and Soil Conditions.—Where deep frost action and excess moisture are prevalent, it is necessary to use greater precaution in having a suitable base than is required in the more arid districts and warmer climates. Many of the roads in California may be treated in the manner described by Mr. Frickstad, without developing the base to greater strength; but if that same base were used in other areas, it would prove a complete failure. It is evident that highway engineers are learning much about the construction of these cheaper types of surface and there is no doubt concerning their great value in developing at least a secondary road system. It is likely they will, as he states, encroach in many places upon the higher

types of surface, yet it is extremely necessary that a thorough study of the local problems involved be made in order to avoid building such types where they are likely to prove failures. This would tend to discredit them in their legitimate and economical use in other places.

Perhaps there has been no phase of highway development that is more of a "missing link," than the need of a low-cost secondary road which will carry a limited amount of heavy traffic and an unlimited amount of light traffic. It is evident that the bill for the high-priced pavements on all highways will retard the proper development of any highway system unnecessarily.

CLAUDE P. OWENS,[13] M. AM. SOC. C. E. (by letter).[13a]—Few highway officials will question the need for a low-cost secondary road that will accommodate a limited heavy traffic and a fairly large volume of light traffic. The problem of keeping up with the fast-growing demand for increased road mileage and improved service with the limited funds available is one that faces every highway official. Traffic is not only increasing in volume, but also in speed. Certain types of surfaces which, a few years ago, were considered ideal and adequate, are now objectionable and unsafe due to the increase in speed and volume of traffic. Furthermore, maintenance engineers are finding their costs mounting to alarming totals such that road surfaces of the loose gravel and stone types cease to be economical. However, construction investments made only a few years ago cannot be discarded until their value has depreciated. Accordingly, any information as to the best methods of salvaging or treating such surfaces to reclaim the invested values, at the same time rendering improved service to the traveling public, is eagerly studied by highway engineers.

At highway conventions the problems and discussions on low-cost roads always occupy a prominent place on the programs. However, in the exchange of papers and discussions proper progress in solving the problems is seriously hampered by the lack of a common vocabulary, understandable by all. The terms introduced in such technical discussions are not clearly defined or are carelessly used. Even when specifications are quoted the discussion is often difficult to interpret. Different chemists have their own pet methods, and engineers their own taught or adopted practices. Viscosities run at 50° cent. are difficult to compare with those run at 60° or 40° cent. However, the U. S. Bureau of Public Roads has inaugurated a movement to bring the many groups together with a view of simplifying and standardizing terms, specifications, and testing methods. When this work is accomplished, the engineer will find it much easier and safer to compare notes, papers, and discussions.

There seems to be a general opinion that the present specifications and tests are not sufficiently comprehensive to be followed clearly or intelligently. Bituminous materials are derived from several sources and fields, each with varying and peculiar characteristics. An oil may contain 60 to 70% of 100-penetration asphalt, but what is the nature of this residue and what were

[13] Engr. of Maintenance, State Highway Dept., Jefferson City, Mo.
[13a] Received by the Secretary December 28, 1931.

the characteristics of the solvent or volatile portion? A cut-back asphalt may contain 25% solvent, but most specifications do not show, nor do common tests reveal, the nature of the solvent. Consequently, when reading or discussing technical papers on bituminous materials used in low-cost road construction, the engineer should do so with "his tongue in his cheek" and with a full understanding of the problem, if he desires to avoid costly errors. It is much better that he approach his problem hopefully, but carefully "feeling his way" with the materials and bitumens at hand.

The development of low-cost bituminous road surfaces in the Western States has been watched with considerable interest. The surface-treatment method is probably the oldest and best known. It requires a thoroughly compacted gravel or stone base. Usually, uncompacted or loose maintenance gravel that is swept from the surface, must be wasted on the shoulders. Most gravel and stone roads are none too thick at best to support surface treatments, and material should be added rather than removed. Furthermore, in compacting and cleaning the surface it is difficult to obtain the thoroughly smooth riding surface now demanded by high-speed traffic. Consequently, it would seem that the surface-treatment method has a rather limited field.

Road Surface Mixes.—The road surface mixes (or plant mixes) seem more nearly to fill all requirements for low-cost bituminous road surfaces. There are several types of mixed-in-place construction, and each type allows the utilization of existing uncompacted road surface metal, or added metal, to build extra thickness and strength to the gravel or stone base. At the same time, these methods permit the leveling and truing up of irregularities of the original gravel or stone surface.

Oil Mix and Fine Aggregate.—In 1928, the Missouri State Highway Department selected a section on U. S. Highway No. 71, in McDonald County, to try out a mixture of road oil and fine aggregate. Methods outlined by C. L. McKesson, M. Am. Soc. C. E., in his "Report of the Co-Operative Work Conducted by the California Highway Department and the U. S. Bureau of Public Roads," were adopted as a guide. The fine aggregate used is known locally as flint, or mine-run chats from the waste piles of the Joplin, Mo., lead fields. Selection was made to obtain as nearly as possible the following specifications:

Screen.	Percentage passing.
¾-in.	95 to 100
No. 4	50 to 80
No. 10	35 to 60
No. 200	5 to 15

The oil used in part met the following specifications:

Specific viscosity (Engler), 50 cu. cm. at 122° Fahr	50 to 65
Percentage of residue, 100 penetration	65 to 75
Percentage of volatilization, in 5 hours, at 160° cent., 20 grammes	15 to 25

Stain tests served as a guide in obtaining the proper mixtures; the mixing was done by blade graders and farm disks; the product was compacted under traffic; and blade graders were used to wipe out traffic marks until the mixture became thoroughly compacted.

The resulting surface texture closely approached sheet asphalt in appearance. Due to the inadequate base some failures occurred during the winter months. The following spring the job was re-worked and more thickness added, the original thickness being about 1½ in. The experimental work was also extended on U. S. Highway No. 71 through Newton and McDonald Counties, using several grades of oil. Oils of lower loss and heavier viscosity were used with fair success.

During construction it was found that to obtain good results, oil should be added in small quantities as the desired total quantity is approached, at the same time avoiding very carefully any excess. The experience with this type of construction has been variable. Aside from weak bases the main difficulty has been due to excess subgrade moisture. Ground-water causes failure apparently because it lifts the oil to the surface of the mix. The bottom portion of the aggregate may be found free of oil, whereas aggregate in the upper portion contains an excess, which results in the familiar corrugated effect. Mixtures of this type are not as adaptable to wet climates as to dry climates.

Road Mix: Retread Type.—The first low-cost bituminous road construction in Missouri was built by the State Highway Department in 1927, on U. S. Highway No. 60, in Greene County. Crushed and screened stone was mixed with several grades of tars and cut-back asphalt to produce a surface thickness of approximately 2 in. Crusher-run stones, 2½ in., 2 in., and 1½ in., in size, with fines out, have all been used. The preference seems to be for the 2-in. size. Stone chips of ¾-in. size are used to fill surface voids. Refined tar of 8–20 viscosity at 40° cent., has been used almost exclusively to prime the old gravel or stone base. Both refined tars and cut-back asphalts are used successfully in the stone mixture.

The equipment in such work consists of a distributor, a 30-h.p., crawler tractor, a multiple-blade maintainer for mixing, a 5-ton roller, a 10-ton roller, and trucks for hauling.

The advantages of this type of construction lie mainly in the fact that it can be built exactly to formula. Aggregate of known quantity and gradation is used. Care is exercised to spread the stone uniformly in a fixed amount per square yard. This is followed by definite alternate applications of bitumen and chips. Excessive mixing is avoided to prevent segregation of stone. An excess of bitumen must be avoided if shoving and corrugations are to be avoided later. In building roads of this type it is easy to add more bitumen if necessary, but very difficult to correct for any excess of bitumen applied. The mixing and leveling process enables the construction of exceptionally smooth-riding surfaces even over fairly rough bases. Large irregularities can be patched before the retread course is laid.

Sufficient time between applications must be allowed for solvents to come off, or for proper curing. The retread type of construction is open in texture

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due to the large aggregates used. This is an advantage in that proper aeration and curing are readily accomplished, which is necessary when cut-backs and tars are used. A seal may be applied later if necessary or desirable, but must not be applied until the bitumens are thoroughly cured.

During the five years since 1927 there have been no failures in this type of construction, except for lack of support, and these have been limited. Such failures are readily repaired by draining the subsoil and adding thickness to the base, and then patching. If failures are extensive the entire surface may be re-worked by leveling and adding more material. If necessary, another course may be built over the original one to obtain the necessary thickness.

Retread: Crushed-Gravel Type.—Where stone is not available at low cost, crushed, screened, and washed gravel may be used with equal success. If the gravel available is fine, crushed, and screened to pass a $1\frac{1}{4}$-in., circular screen, tar has been found to be preferable. However, when the finer aggregates are used, it is necessary, of course, to increase the percentage of bitumen used.

Road Oil Mats.—During the two years, 1930–31, limited experiments have been conducted in compacting gravel and stone surfaces by the use of road oil. In modern gravel road construction, the road metal is windrowed along the shoulder and fed on to the surface blades as it is compacted by traffic. Maintenance replacement gravel is also placed in windrows and gradually bladed on to the surface. During dry weather, traffic loosens the gravel, and blades are used to return the surplus loose gravel to windrow, where it remains until wet weather will allow its return to the surface for compaction.

During experiments with road oil as a dust palliative, it was found that an excess of oil formed a mat. Under favorable conditions these oil mats gave good service. Consequently, road oil meeting the following specifications is being used experimentally for building oil gravel mats:

	Oil No. 2.	Oil No. 3.
Specific gravity, 60° Fahr./60° Fahr., not less than.....	0.930	0.960
Furol viscosity, at 122° Fahr..	125 to 225	225 to 380
Flash point, not less than.....	190° Fahr.	194° Fahr.
Bitumen soluble in CS_2, not less than	99.5	99.5
Volatilization, 5 hours at 325° Fahr., 20 grammes	0 to 18%	0 to 18%
Percentage of residue, 100 penetration	55 to 75	65 to 80
Ductility residue, not less than.	50 cm.	50 cm.

The No. 2 oil is best for first or prime application, whereas the No. 3 oil is used for mixing with the gravel. The mixture is compacted under traffic and kept smooth by motor or blade graders until thoroughly compacted.

This method calls for an excess of oil, which is objectionable to traffic during the process of curing. During the first 2-week period the mixture is "messy," and traffic has just cause for complaint. Recently, a trial was

made of the use of powdered asphalt, and early indications are most promising. Powdered asphalt shipped in paper bags was distributed through a lime spreader at the rate of ½ lb. per sq. yd. on a freshly oiled gravel surface. The powdered asphalt has a tendency to hasten the setting period, and possibly adds strength and body to the oil.

Width, Thickness, and Cost.—It would appear that any road carrying sufficient traffic to warrant these surfaces would also justify minimum widths of 20 ft. In some cases greater widths may be justified on account of their water-proofing value to shoulders and edges alone. Experience in Missouri proves that a 2-in. thickness is entirely sufficient to resist shear and scaling where the sub-base support is at all adequate. Greater thickness represents unnecessary waste of funds and possibly invites some trouble due to the difficulty of handling greater quantities in road mixing, and of proper curing which is a most important feature not to be overlooked, especially when tars and cut-backs are used with finer aggregates.

It is difficult to predetermine the strength of an old gravel or stone base. Slight movements under loads during frost or wet seasons are likely to be unobserved on a well-maintained gravel road. These movements are quite likely to become readily noticeable after the bituminous surface is built. Consequently, the type of bituminous construction first used should be of low cost, easy to reconstruct, easy to repair, and easy to add to, if necessary. As a general rule, base failures are confined to small isolated places or to particular portions of the projects. Funds conserved in the building of minimum thickness may be used for special treatment of these places.

Furthermore, in Missouri there is a marked tendency for any road (pavements included) to lose its true cross-section and profile under the action of climatic changes and the pounding of traffic. Therefore, it seems logical to censerve funds by building the required minimum thickness and to use the money so conserved at a later date to add extra thickness where necessary to obtain greater strength or to improve the cross-section and profile, or both.

Cost, exclusive of stone and gravel, aggregate which vary greatly, range from $750 to $2 500 per mile for 20-ft. widths, depending upon the bitumens used. Oil-graveled mats, 1½ to 2 in. thick, cost from $750 to $1 000 per mile for oil, oil application, and mixing. Stone retread costs from $1 800 to $2 200 per mile, which includes cut-back or tar applied, spreading stone, mixing, and rolling. Seal coats of ⅓-gal. application and 15 lb. of sand or chips per sq. yd., cost from $600 to $900 per mile additional. Gravel retread requires more bitumen on account of the greater percentage of fine materials.

If the cost of stone or gravel aggregates is charged against maintenance gravel replacement funds, the foregoing costs are net. Since these treatments conserve road-surface material from loss due to action of traffic and water, further reductions might be deemed proper.

Plant Mixing.—The experience in Missouri with plant mixes on low-cost bituminous road construction has been limited. Advocates of plant mixes claim certain advantages of control over mixtures not obtainable in road mixes. Possibly this is true with certain plants, but the contrary is only too true with many plant mixers. Where better control is obtained in the

use of a plant mix without any appreciable increase in cost of production, a plant is justified. To justify its place in the highway program, the low-cost bituminous surface must remain in the low-cost field in fact and not in name only. Elaborate methods and expensive procedures must be avoided, especially if no material benefits are added. This type of construction does not justify frills.

On the other hand, low-cost bituminous road construction should not be haphazard or careless. Care and skill may be used without involving expensive methods.

Where tars and cut-backs are used, the engineer should remember the nature of his materials. These materials contain certain solvents that must come off and allow the body material to stiffen after mixing. Therefore, certain aeration is necessary, which is accomplished in a large measure during the turning and mixing process in the road-mix method.

New Construction.—The highway engineer should not confuse low-cost bituminous surfaces with high type pavements; they are poor substitutes. However, the low-cost bituminous surface has its own field, which lies between the traffic-bound stone or gravel road and the high type pavement designed to carry the combined traffic of high-speed and heavy vehicles.

CARLTON N. CONNER,[14] M. AM. SOC. C. E. (by letter).[14a]—All road builders are indebted to Mr. Frickstad for his successful efforts in developing low-cost roads on the Pacific Coast. Research and practice developed by him and his associates have gone a long way toward solving the problem of building thousands of miles of all-weather roads in the eleven Western and Mountain States at a total cost for surfacing, including base and bituminous surface, of less than $10 000 per mile.

Low-cost construction in the United States has been made possible by the extensive use of tars and asphalts as binders, local materials as aggregates, and improved equipment for the actual work. Developments have been so rapid that the time is now ripe for simplification of methods and specifications, and the standardization of tests for materials, in fact, a consolidation of the rapid and successful advances already made. A concerted effort at this time is particularly important because unmistakable signs and trends show that construction and maintenance of low-cost roads will more completely occupy the attention of road builders for several years to come.

To assist in clarifying the field for standardization and to focus attention on the principal types of low-cost roads, the writer invites attention to the following discussion which is based on present conditions.

Low-cost surfaces, of which there are large mileages, or which are growing in favor, are commonly grouped as: (1) Untreated surfaces; (2) non-bituminous surface treatments; (3) bituminous surface treatments; (4) bituminous road mixes; and, (5) bituminous plant mixes.

In 1931, the State highway systems had 169 000 miles of these low-type surfaces and 95 000 miles of high-type surfacing. Table 3 shows the trends in construction and maintenance of untreated and treated roads.

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[14] Engr.-Executive, Am. Road Builders' Assoc., Washington, D. C.

[14a] Received by the Secretary January 7, 1932.

Untreated Surfaces.—Untreated surfaces are those which consist of mineral aggregates held together by the binding, compacting, or interlocking qualities of the materials themselves, or by the addition of mineral binder, such as clay. These untreated types include sand-clays, gravels, stone, chert, shale, and others less commonly used.

TABLE 3.—STATE HIGHWAY SYSTEMS: TRENDS IN LOW-COST SURFACES BY MILES

State	Built during 1930, untreated and treated	Proposed 1931 Construction		Expected 1932 Construction		Expected 1932 Maintenance	
		Untreated	Treated	Untreated	Treated	Untreated	Treated
Alabama	337	125	46	...	...	...	...
Arizona	309	...	295	...	...	...	...
Arkansas	1 378	200	200	200	100	5 450	300
California	317	490*	...	...	119	...	2 050
Colorado	175	200	75	150	150	1 500	500
Connecticut	124	...	100	...	...	...	...
Delaware	19	20	...	10	5	...	38
Florida	16	30	...	...	100	235	3 175
Georgia	47	70	80	...	125	500	825
Idaho	576	500	200	50	600	1 694	906
Illinois	...	30	...	...	...	...	...
Indiana	156	150	...	...	150	1 800	1 600
Iowa	247	240	...	...	...	...	...
Kansas	853	700	50	200	400	4 000	900
Kentucky	692	459*	...	375	50	2 400	1 650
Louisiana	1 015	1 300	500	...	...	...	...
Maine	72	8	76	400	35	1 690	2 100
Maryland	71	50	65	35	40	160	1 600
Massachusetts	...	...	...	...	95	...	255
Michigan	86	77	15	...	...	...	...
Minnesota	640	150	650	...	500	3 122	1 900
Missouri	893	1 000	...	1 500	50	3 180	150
Mississippi	...	...	...	...	...	...	...
Montana	195	750	...	...	...	...	...
Nebraska	489	900	25	1 000	100	6 200	150
Nevada	456	330	...	250	275	1 064	722
New Hampshire	171	50	50	...	90	100	1 940
New Jersey	1	10	...	...	...	...	...
New Mexico	811	202	198	133	172	1 900	600
New York	144	75	...	...	...	...	...
North Carolina	303	100	100	...	...	...	...
North Dakota	454	400	12	700	100	4 319	374
Ohio	154	100	200	...	...	...	...
Oklahoma	1 020	300	1 000	...	...	...	...
Oregon	254	100	150	150	100	1 050	1 650
Pennsylvania	610	...	...	...	...	...	...
Rhode Island	33	...	35	...	...	...	132
South Carolina	120	75	325	150	325	2 500	900
South Dakota	520	200	300	600	300	4 050	200
Tennessee	600	...	...	...	...	...	...
Texas	450	300	400	210	426	5 274	4 069
Utah	245	214	215	...	...	...	...
Vermont	187	100	100	...	...	...	...
Virginia	357	335*	...	...	...	...	...
Washington	743	256	157	203	...	380	...
West Virginia	284	125	125	...	...	...	...
Wisconsin	513	260	120	400	200	2 900	3 000
Wyoming	364	425	520	...	...	...	...

* Treated and untreated.

For the purpose of this discussion and to be consistent with tabulations already prepared by the American Association of State Highway Officials and the U. S. Bureau of Public Roads, untreated surfaces are grouped as: (*a*) Sand-clay and top-soil; (*b*) gravel, chert, shale, etc.; (*c*) water-bound macadam; and (*d*) miscellaneous.

Sand-Clay and Top-Soil.—These types of road surfaces usually consist of clay, silt, fine sand, and coarse sand, with a total sand content of from 65 to 85 per cent. They seldom contain particles larger than $\frac{3}{4}$ in. in size.

The States having from 800 to 3 000 miles of these surfaces are Alabama, Florida, Georgia, Kansas, North Carolina, South Carolina, Texas, and Vermont. In 1930, Alabama, South Carolina, and Virginia laid between 100 and 200 miles each, while Kansas surfaced more than 1 500 miles with sand-clay.

Gravel, Chert, Shale, etc.—Available tabulated classifications do not list separately the miles of treated and untreated roads, but the total at the end of 1930 was 107 000 miles. Only fourteen States have less than 1 000 miles of the gravel, chert, etc., types, while twenty-two have from 2 000 to 6 000 miles of each.

Gravel is used for surfacing in all States, and there is a marked trend toward using gravel passing the 1-in. screen with crushed particles favored. Until recently, traffic-bound surfaces of gravel, stone, or slag were used principally in Ohio, Indiana, Tennessee, Nebraska, and Iowa. This type has proved particularly economical and serviceable, and is meeting with favor in Kentucky, North Carolina, and elsewhere.

Traffic-bound surfaces consist of a layer of clean crushed stone, slag, and crushed or screened gravel evenly spread on a prepared sub-grade. This layer is kept smooth by constant blading. Traffic gradually compacts it from the sub-grade upward. Periodically, new aggregates are added, under traffic, until a surface of suitable thickness is built up.

Macadam.—Macadams of crushed stone, slag, crushed gravel, and Florida lime rock are commonly laid in one course, and, less frequently, in two courses, to form a well compacted surfacing, suitable for improvement with bituminous materials.

Roads of the macadam type, and specifications for them show notable differences in requirements for quality and size of aggregates, thickness of section, and details of construction, but a study of the specifications shows they are written to secure good quality and a practical gradation of the available local materials and that softer stone is permitted in base courses which, later, will be surface treated. In 1930, the State highway departments built 1 300 miles of this type, as compared with a total of 13 000 miles of gravel, chert, and similar types.

The States which have more than 1 000 miles of macadam types in their highway systems are Florida, Kentucky, Maryland, New York, Ohio, Pennsylvania, Tennessee, and Virginia. Kentucky leads the list with about 4 000 miles. Kentucky also built 400 miles of macadam in 1930, which was more than twice that built by any other State; but Florida, Pennsylvania, West Virginia, and Wisconsin each built more than 100 miles.

Miscellaneous Untreated Surfaces.— These include surfaces of various kinds of aggregate which are laid in one or two courses as a type of water or roller-bound macadam, or they are placed in loose layers of 1-in. or 2-in. thicknesses to form a traffic-bound surface.

Materials for them include scoria, decomposed (or disintegrated) granite, stone screenings, mine "chats," iron ore, top-soil, granulated slag, pea gravel, stamp sand, shell, and volcanic cinders.

The use of hand-napped field stone as a base, covered by crushed stone, slag, or gravel, has met with success in Pennsylvania in the surfacing of 1 700 miles of the 20 000-mile program of low-cost roads.

Non-Bituminous Surface Treatments.—The purpose of these treatments with calcium chloride or lignin binder is to lay dust and reduce the loss of aggregates.

Calcium chloride is commonly applied to the road surface in flake or powder form. It has been used quite extensively in Michigan, New York, Vermont, Maine, and, to a lesser degree, in other States east of the Mississippi River. It has not been successful in States having relatively low humidity.

Lignin binder is a liquid concentrate which when mixed with equal parts of water is sprinkled on a road surface. Lignin binders have been used principally in New Jersey and New York.

Bituminous Surface Treatments.—Bituminous surface treatments as commonly designated are two or three applications of bitumen which are covered by sand, gravel, stone, or slag. These successive applications of bitumen and cover make a thin bituminous mat from $\frac{1}{2}$ in. to 1 in. in thickness, which is well bonded to the base; when more than 1 in. thick these dragged or mixed surfaces should be designated as road-mix. The recent and successful innovation of dragging surface treatments which are less than 1 in. in thickness, during construction, makes them approach the road-mix type in texture, appearance, thickness, and smooth-riding qualities.

Bituminous surface treatments have been extensively used in the North Atlantic and South Atlantic States, Ohio, Indiana, Kentucky, and Florida on macadams and, in more recent years, on gravels, sand-clays, and traffic-bound surfaces.

Applications of light asphaltic oils on earth roads have met with little success as compared with heavier bitumens on macadams and other substantial bases. In certain instances on selected soils which were stabilized naturally or artificially by coarse granular particles, such as coarse sand or stone screenings, they have improved the road surface.

Bituminous Road Mixes.—The term, "road mix," is used to designate the surface obtained by mechanical mixing of aggregates and bitumen directly on the road itself. "Mixed-in-place" is another term widely used and equally descriptive. Road mixes are commonly from 1 in. to 3 in. in thickness and are not to be confused with plant mixes or with dragged-surface treatments.

Although there are other types of bituminous road mix, such as the fine aggregate types, which use sand-clay or blow sand for aggregate, most of them are in the experimental stage. There are two distinct types of road mix which are identified by the characteristics of their aggregates. These types are the "graded aggregate" and the "coarse aggregate."

The "graded aggregate" type is usually composed of gravel or stone and sand uniformly graded from a maximum size of about 1 in. to dust. These

aggregates are as found in the road, as added to the road surface, or as added and mixed with aggregate already in the road surface. The type is used extensively in the Western States, the Southwest, and parts of the Mississippi and Ohio Valleys, where it is known by such names as "oil mix," "oil process," and other designations.

The "coarse aggregate" type, or "macadam aggregate" type, as the name implies, is composed of aggregates the characteristics of which are similar to those commonly used in macadams. They must be hard, durable, and preferably angular, such as crushed stone, slag, and crushed gravel. The aggregates are uniformly graded from those passing the $1\frac{1}{2}$-in. screen to those retained on the $\frac{1}{4}$ or $\frac{3}{8}$-in. screens. This type, also widely known as "re-tread," is built principally east of the Mississippi River and west of the Atlantic Coast States. Indiana, Pennsylvania, Kentucky, and Tennessee have each several hundred miles.

Still another type of road mix should be mentioned which for convenience may be termed "road-mix—fine aggregate" type. This type includes bitumen mixed on the road with sand-clays, sand-gravels, fine clay-gravel, or blow sands, the aggregates in which are much finer and commonly of poorer quality than those of the "graded" or "coarse aggregate" types.

Work on such roads has been confined to limited areas or has been of an experimental nature. Experimental work on them is still in progress, and they offer a potential field for needed development. Attention is invited to experimental work in South Carolina, Florida, North Carolina, California, Nevada, and Minnesota, and to extensive usage in Long Island, New York.

Bituminous Plant Mixes.—"Plant mix" means the utilization of fixed or portable plants to prepare low-cost surfacing mixtures of bitumen and aggregates. The aggregates for plant mixes in low-cost roads are commonly those of the graded type already described, but they may be of the coarser sizes.

The object of plant mixing is to insure closer control of proportions and uniformity of mix than is possible by mixing in place on the road surface. As a result any plant mixing produces more fully coated aggregates than road mixing, but the requirements in specifications should not be so severe as to discourage contractors and manufacturers from developing plant mixes at low cost.

Whether or not the extra cost of plant mixing is justifiable has not been fully established, but experience and trends indicate its growing popularity where cost of aggregates and hauling is low, or where the road needs surfacing with aggregates which are of better quality than those found in the existing surface.

HAROLD L. TILTON,[15] ASSOC. M. AM. SOC. C. E. (by letter).[15a]—To any one outside the Western States this paper emphasizes the fact that while the general methods of construction are quite similar throughout the country, the engineer in each section must adapt these methods and principles to meet

[15] With Vermont Dept. of Highways, Montpelier, Vt.

[15a] Received by the Secretary January 19, 1932.

the climatic conditions to be encountered in his State and utilize to the best advantage the particular aggregates and bituminous binders that are available to him.

In New England, the use of a light fuel oil which would tend to emulsify in the presence of moisture, would be out of the question in the construction of a road that would be satisfactory at all times of the year. Rain occurs often and at any time. In the winter, the frost penetrates deeply. In the spring, it comes out of the sub-base while the atmospheric temperature is still so low that a bituminous surface lacks the plasticity that it attains later in the season. Aggregates containing fillers, such as silt and argillaceous materials, which possess a greater affinity for water than for bitumen, must be avoided. Surfaces built with such materials would be summer roads only. Engineers in New England are forced to use the heavier oils, and it is interesting to note that the author states that the trend in the West is in the same direction.

For dust-laying purposes only the Vermont Department of Highways ordinarily uses calcium chloride. Low-cost bituminous roads in Vermont are an intermediate type of surface designed to meet traffic conditions where the chloride-treated surface cannot be maintained satisfactorily, and yet where the cost of a high type of pavement is not warranted. One problem is common throughout the country. There are many miles of secondary roads badly in need of an improved surface and there are comparatively few dollars available for the needed improvement. The low-cost bituminous road seems likely to be the solution of that problem.

The use of the drag, the retread mixer, and the blade grader, together with the development of emulsions, retread tars, and cut-back asphalts, has revolutionized the art of constructing and maintaining low-cost bituminous surfaces. In the past most of them were far from being smooth-riding roads. This was due to the fact that hand labor was used so largely in spreading the aggregates. To-day, it is possible to build low-cost bituminous roads by machinery with a minimum of hand labor, that are very smooth riding. Furthermore, by similar methods, they can be kept smooth riding, which is a highly essential factor to be considered.

Mr. Frickstad's statement that the principal failures continue to arise from faulty support, due to deficiency in thickness of metal or stability of sub-grade, deserves careful consideration.

It is quite evident that a bituminous-treated surface of gravel or broken stone 2 or 3 in. thick, has little inherent resistance to deflection. Such a surface will retain its true cross-section and grade just as long as it is unyieldingly supported by the foundation on which it rests. An adequate and well-drained sub-base is absolutely essential to the success of such a road. It is a waste of money to construct a bituminous treated surface on a sub-grade that will fail to support it.

The importance of a sufficient thickness of metal is the reason why the mixed-in-place method is used on gravel surfaces in Vermont, instead of surface treatment. By the former method an adequate and fairly uniform thickness of metal is obtained at once, which, by the latter method, is usually

secured only after several successive years of treatment. The depth of surface obtained by the penetration method varies according to differences in the grading of the gravel, and most of the aggregate particles have a heavier coating of bitumen than is necessary to make them adhere to adjoining particles. A familiar comparable fact is that a thin film of glue will make two pieces of wood adhere more strongly than a thick film. In the mixed-in-place method, the particles are coated with only enough bitumen to bind them together properly and a fairly uniform depth is obtained.

Thus far, experience has led highway engineers in Vermont to prefer the use of tar in the construction of gravel surfaces and the use of cut-back asphalts and emulsions in the construction of broken-stone surfaces.

The author states that "the grading of aggregate has not tended to become standardized, which confirms the opinion that a wide range is acceptable for oil mixing." This bears out experience in tar mixing in Vermont, where aggregates have been used with a range in maximum size from $\frac{3}{4}$ in. to 2 in. and containing a variation in fineness modulus of the sand from 2.60 to 4.10. Good results have been obtained with the stone content of the aggregate ranging from 40 to 75 per cent. An aggregate containing about 70% of stone produces the most stable and best wearing gravel surface.

In oil mixing the author states that the proportion passing the 200-mesh sieve is an important element. In tar mixing its importance lies in the desirability of its elimination. An excessive quantity of material passing not only the 200-mesh, but also the 100-mesh, sieve, almost invariably is present in surfaces that have failed from lack of stability. Particularly is this true when the fine material is argillaceous, which type possesses a greater affinity for water than for bituminous binder.

A broken-stone mixed-in-place surface is unquestionably stable. Its stability is due to three factors, namely, the keying and interlocking of the angular pieces of stone, the grading of the stone, and the binding qualities of the bitumen. In such a surface there is a considerable percentage of voids both large and small.

A gravel surface containing a coarse sand may contain a small percentage of voids such as would be caused by the absence of material passing the 100-mesh sieve. It may possess high stability due to the fact that so many of the stone and sand particles are quite angular. When the gravel is crushed, this characteristic is even more pronounced. A washed, crushed gravel surface containing 70% of stone is not greatly inferior to a broken-stone road when the stones in each case are of an equal degree of hardness and the proper bitumen is used in each surface. The removal of the very fine material is a decided advantage. As in the broken-stone road, a limited excess of bitumen can be taken care of by the voids. On the other hand, an excess of bitumen with a gravel in which all the voids are already filled with fine material, will be quite likely to develop instability.

A sharp sand, with a fineness modulus of not less than 2.65 and preferably about 3.50, with a color rating of not more than 2, and containing not more than 5% passing the 100-mesh sieve, is most satisfactory for tar mixing. The

more closely the characteristics of a gravel surface approach those of a broken-stone surface, the more stable it is likely to be, and the less maintenance it will require.

The author notes the presence of an excess of 200-mesh material, usually clay, whenever failures occur from moisture conditions. This appears to be almost invariably the case. On one project in Vermont the gravel was washed to remove the fine material of an argillaceous nature and the results were decidedly satisfactory. Clay, silt, and loam are no more desirable when mixed with bitumen than when mixed with cement.

The wear of the gravel preferably should be not more than 20%, although gravel with 24% wear may be used if no harder gravel is available. The best results have been obtained by crushing and screening the gravel, separating the sand, and re-combining them in definite proportions. For most gravels a tar of 20 viscosity is advisable for the mix and one of 40 viscosity for the seal, although the lighter tar has produced a good seal. On one project a retread tar of 65 viscosity was used for the seal. It was covered with a coarse hard gravel of 1¼-in. maximum size and dragged and rolled. This produced an excellent non-skid surface. Two additional requirements in tar specifications are now under consideration by the New England States. A requirement for specific gravity will eliminate an excess of water-gas tar in the blending. A lower limit for the softening-point requirement should eliminate a tar unsuitable for cold wet weather conditions. The author's estimate for bitumen, of ½ gal. per sq. yd. per in. of finished thickness (varying, of course, with the grading of the aggregate), seems to be the general experience.

For gravel mixing a 20-ft. drag has been designed, with blades in zigzag arrangement, which has proved the fastest and most efficient mixer. A grader with approximately the same wheel-base is a necessary supplementary piece of equipment for the final shaping.

It is advisable in the construction of both gravel and broken-stone surfaces to apply a prime coat of bitumen to the sub-grade about a week in advance of the surface construction. This serves the double purpose of eliminating the dust nuisance to the public during construction and prevents the inclusion in the mix of undesirable fine and unclean materials from the sub-base.

Broken-stone surfaces may be constructed successfully with an aggregate not larger than 2½ in., which materially increases plant production and decreases costs. It is desirable, as in hot-penetration macadam, to have the maximum size of stone nearly equal to the depth of surface. For the seal, a stone graded from ¼ to ¾ in. is used. In a New England climate the heavier cut-back asphalts are the most satisfactory. The best results have been obtained with the use of a so-called benzol cut-back having a viscosity (Engler, 50° cent.) of 150 to 175, distillate at 680° Fahr., of about 20%, and penetration of the residue of 90 to 100. The surface obtained with this asphalt compares favorably with that produced by the use of a hot-penetration asphalt in macadam construction. In building broken-stone mixed-in-

place surfaces, a grader with a wheel-base of 18 ft., or more, is a satisfactory mixing and smoothing agent. The seal is smoothed by the use of the drag or the retread mixer.

Good results have been obtained with the use of a penetration emulsion, the stone being smoothed by a grader, and the emulsion applied in three applications, each properly keyed and rolled. The emulsion has one property very definitely in its favor in a New England climate; it may be used with a wet aggregate. The time lost in construction in Vermont during 1931, while waiting for aggregates to dry, represented no small sum of money.

The average cost of gravel mixed-in-place surfaces constructed in 1931, including the cost of the new gravel, was about $3 600 per mile. On most projects this gravel had to be hauled from 3 to 8 miles. The cost of mixing and rolling alone was about $200 per mile.

The average cost of broken-stone mixed-in-place surfaces constructed in 1931, including the cost of the stone, was about $5 600 per mile. All this work was done by State forces.

The construction of low-cost bituminous roads necessarily means that local aggregates should be used to as great an extent as possible. This requires study as to the best methods of combining them with the bitumens that are available in that particular section of the country.

L. I. HEWES,[16] M. AM. SOC. C. E. (by letter).[16a]—That low cost or intermediate types of bituminous road have made substantial contribution to the road developments of the West cannot be questioned. On December 31, 1930, practically 20% of the Federal Aid highway system of the Western States consisted of a surface-treated, plant-mix, or road-mix construction, described by Mr. Frickstad.

Materials.—Aggregate of surprisingly wide range of grading continues to be successful in road mix under varying conditions and with different available local materials. General Western practice in 1932 is to use aggregates of 1-in. maximum size to dust, and a 2½-in. minimum compacted thickness of road-mix or plant-mix surface courses. Oils of higher viscosity are meeting with increased favor, especially where moisture influence is expected. When light oils are used, some engineers are placing a seal coat of heavier oil, with sand or screenings, two weeks or more after completing the road mix. There has been a limited use of "armor coat" treatment approximately ½ in. to ¾ in. thick placed on previously constructed road mixes. All surface treatment requires great care in distribution of material and cannot wholly correct imperfections in the under surface. Cut-back plant mixes are being tried, especially in California. It should be borne in mind that the use of cut-back asphalts or asphaltic oil of higher viscosity increases the cost of scarifying or remixing whenever required during maintenance.

Construction.—No formula yet proposed for the control of the oil and aggregate has received general endorsement. Advance laboratory analyses and tests of the aggregates and bituminous materials will usually furnish

[16] Deputy Chf. Engr., U. S. Bureau of Public Roads, San Francisco, Calif.

[16a] Received by the Secretary February 19, 1932

valuable indications. It is well to know in advance whether the aggregate is highly absorptive, in order to avoid a lean mix which might ravel. Bituminous mixes that would give unsatisfactory service under unfavorable moisture conditions because of characteristics of the fine material, are often detected in the laboratory by a swell test.

A moist or loose uncompacted base is a frequent cause of distress. Many road-mix or plant-mix projects, with aggregate ranging from 1 in. to 200-mesh material, have shown failures traceable to excess moisture in the materials during the mixing or prior to the final spread and compaction. Most engineers think that an excess of 3% moisture will prevent satisfactory compaction on the road, or possibly cause failure. Accordingly, the practice of heating aggregates in plant mixing is increasing.

Adherents of plant mix claim better control and the possibility of using materials of higher viscosity. Its field is limited, necessarily, to projects on which the mineral aggregates to be treated have not been placed on the road. Plant-mixed material that becomes wet before spreading should be bladed until the condition is corrected. This operation is also frequently used to increase the evaporation of the solvent in cut-back mixes. Contractors do not resort to rolling ordinarily. However, it is advantageous under unfavorable traffic conditions, for compacting the edges, also when heavy rains are expected, or when relatively quick-hardening asphalt products are used in late season work.

Equipment manufacturers are developing several types of self-propelled machines designed to mix the materials while moving along the road. There may be a field for further experimentation in this direction.

Oil road construction commends itself because of low first cost and cheap maintenance. Expensive refinements in plant mixing and road mixing should be avoided unless the results are certain to be commensurate with the increased cost. Unwarranted expenditures, especially when expensive importation of aggregates is involved, also must be watched.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

LAKE CHAMPLAIN BRIDGE

Discussion

BY MESSRS. ROBERT W. ABBETT, HENRY W. TROELSCH, JACOB FELD, AND CLARENCE W. HUDSON

ROBERT W. ABBETT,[7] ASSOC. M. AM. SOC. C. E. (by letter).[7a]—The author is to be commended for his thoroughness and originality in planning the Lake Champlain Bridge. It stands as another monument to the death of what has been a bugaboo to American bridge engineers for the past century. Continuity in bridge construction has been scrupulously avoided in design and vigorously attacked by many of the most eminent designers for several reasons; notably because it is statically indeterminate and because a slight pier settlement has been thought to introduce critical stresses in various parts of the structure. That the weakness and inaccuracy of these reasons have been recognized is proved by the increasing popularity of this type of construction, and by the large number of continuous trusses that have been built since Gustav Lindenthal, Hon. M., Am. Soc. C. E., proposed the Sciotoville Bridge.

The layout of the Lake Champlain Bridge is most ingenious. The writer can think of no other arrangement which would have provided the necessary clearance at mid-span with such economy in approaches. In the high-level viaduct from Newark to Jersey City, N. J., a somewhat similar layout was necessary for the river spans. Here, however, the cantilever principle has been used with comparatively long suspended spans. Since the approach conditions for the two sites are quite similar, the writer cannot hold with the author in his unsupported statement that a short suspended span was necessary in the cantilever type. An economic comparison between the continuous truss, as built, and a cantilever truss with a longer suspended span similar to the Hackensack and Passaic River Bridges, would be interesting.

It appears to the writer that the method of consistent deflections might have been utilized in the stress analysis to some advantage over the method

NOTE.—The paper by Charles M. Spofford, M. Am. Soc. C. E., was presented at the meeting of the Structural Division, Boston, Mass., October 10, 1929, and published in December, 1931, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[7] Instr., Civ. Eng., Sheffield Scientific School, Yale Univ., New Haven, Conn.

[7a] Received by the Secretary December 21, 1931.

of least work. This method seems to be more simple and less laborious. The elastic curve for the truss may be found by removing the redundant, R_L, and drawing the deflection diagram for a unit load at its point of application. The influence line for the stress in any member in an end span is easily constructed using the elastic curve as a foundation. Thus, the influence line for Member, U_9–U_{11} is constructed by simply drawing a straight line across the

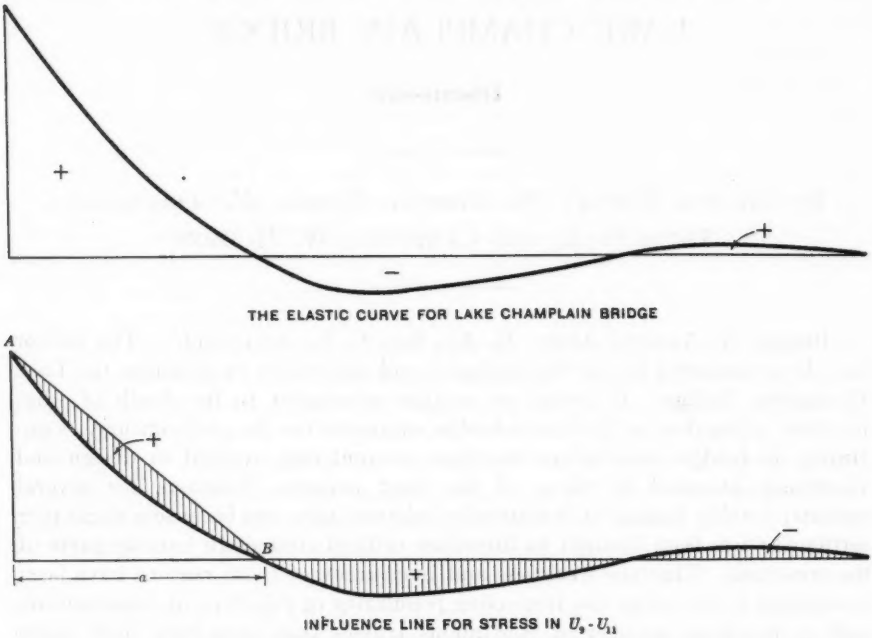

FIG. 14.—INFLUENCE LINE FOR LAKE CHAMPLAIN BRIDGE

elastic curve from A to B (Fig. 14) and multiplying each ordinate of the resulting diagram by $-\frac{a}{r}$, in which, $r =$ the lever arm of Member U_9–U_{11} about its center of moments in the truss, and $a =$ the distance of R_L from the same center of moments. For members in the central span the construction of influence lines is a more lengthy process, but fundamentally it is no more complicated.

HENRY W. TROELSCH,[8] M. AM. SOC. C. E. (by letter).[8a]—Some of the circumstances encountered in carrying out the field work in connection with the erection of this bridge should be of general interest.

The spans erected on falsework were supported at alternate panel points on towers, each of which consisted of two bents braced together. The erection travelers, running on the floor of the bridge, were used to place the false-

[8] Designing Engr., Am. Bridge Co., New York, N. Y.

[8a] Received by the Secretary January 16, 1932.

work and to erect the spans in one pass, working from the shore to the middle of the through span.

The mud of the lake bottom was very soft, and no difficulty was encountered in driving piles to a bearing on the underlying rock. In some cases, however, the rock was 100 ft. below the water level, and piles, 140 ft. long, were required. The piles consisted of 12 by 12-in. timbers, and were spliced by four angles, 4 by 4 in. by $\frac{5}{16}$ in., fastened with $\frac{3}{4}$-in. through bolts. Squared timber piles, being free from the initial curvature usually present in round piles, are less subject to bending from column action. They also have a larger area than round piles of the usual dimensions, and can carry a greater load, especially when they are driven to bearing on rock. They can usually be withdrawn, and have a high salvage value.

The piles were driven by a steam hammer suspended from the boom of the traveler. No leads were used; instead, wooden guides were lashed across cables extending from the completed part of the span to the next pier.

The closing of the through span was simplified by the shape of the structure and by the presence of a roller shoe at the north intermediate support, L_{24}. The end supports, at L_0 and L_{34}, were erected 20 in. below their normal position, which was equivalent to rotating the two halves of the structure about the intermediate supports, L_{10} and L_{24}. Since the closing joints, U_{17} and L_{18}, were well above the centers of rotation, the rotation caused a separation of the two halves of the structure at these points, leaving a gap at each closing joint. The gap in the upper chord was about an inch larger than that in the lower chord.

The closing operation consisted of rolling the north half of the structure southward until the gap in the lower chord was closed, and then jacking up the end supports, L_0 and L_{34}, until the upper chord was closed. The closure was thus accomplished without temporary or adjustable members, such as are ordinarily used in closing cantilever bridges, and without drilling any of the connections in the field. No special equipment was required other than four 300-ton hydraulic jacks.

JACOB FELD,[9] ASSOC. M. AM. SOC. C. E. (by letter).[9a]—The writer is chiefly interested in the description of the substructure design and construction. Although given in great detail there are several questions which might be answered in the closing discussion to complete the record.

The contractors were asked to bid on the construction of all piers by the open coffer-dam process with an alternate choice of constructing the channel piers by the pneumatic process. In accordance with information shown by the borings, with approximately 50 to 60 ft. of fine uniform soft materials to penetrate, it seems that a design of precast concrete or steel-shell caisson might have proved just as economical, if not more so, than the open coffer-dam. Such a design would have given a better concrete surface than can be guaranteed to result from concrete placed under water in spite of all precautions that were taken.

[9] Cons. Engr., New York, N. Y.

[9a] Received by the Secretary February 4, 1932.

There is no mention of any special precautions in protecting the surface of the piers at the water-line to take care of ice pressure and wear during the winter seasons. Undoubtedly, this item was considered in the design, and the omission of special protection for the concrete, an unusual procedure, must have resulted from definite reasons. It would be interesting to have those arguments on record.

An item often overlooked in the preparation of construction procedure for such piers is the accumulation of laitance on the top of each pour of under-water concrete. In Piers 6 and 7, the total height of under-water concrete was approximately 60 ft. A considerable thickness of soft material must have been exposed after unwatering. The chemical tests of this soft material in a number of projects has shown that the quantity of cement removed from the concrete is between 10 and 15%, so that the total amount of soft material which can be expected is about one-twenty-seventh the total height. In these piers the amount of soft material on top of the under-water concrete should have been somewhat more than 2 ft. It would be interesting to know what depth of soft material was actually removed from the top of under-water concrete, and whether any tests were made as to the strength of the concrete below the removed material.

CLARENCE W. HUDSON,[10] M. AM. SOC. C. E. (by letter).[10a]—This paper is valuable for several reasons. It gives a comprehensive description of the engineer's service to his client from the inception of the project to its completion; it describes the work of the engineers preliminary to their actual engineering service; it begins with certain traffic statistics, from which estimates of traffic on the proposed bridge and of the financial returns from such traffic were made; and the amount of money that could justifiably be spent for the construction of a bridge was determinable from the estimated return from traffic.

The following comments apply to the various subjects described by Professor Spofford:

(*a*) The author describes the service of the engineers after the financial feasibility of the project was established.

(*b*) The description of the field survey is very concise and yet comprehensive and clear.

(*c*) The engineer's conclusions with respect to the type and details of the structure to be adopted, were sound.

(*d*) The reasons given for the elimination of simply supported and cantilever type truss spans, from their design for the central portion of the structure, the writer believes are correct.

(*e*) The writer does not believe, however, that the difference in cost due to using simple truss spans, cantilever spans, or continuous spans, for the principal part of a structure, would, in general, be large enough in itself to be a controlling factor as against a better appearing or more satisfactory structure inherent in any one of the types, for use in a definite location.

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[10] Cons. Engr.; Pres., C. W. Hudson, Inc., New York, N. Y.

[10a] Received by the Secretary February 9, 1932.

(*f*) The description of the method of building the deep piers is excellent and the word or two with reference to the engineer's effort to eliminate the harmful effect of laitance (which the writer has always found to be existent) in concrete deposited under water, is evidently good.

(*g*) The writer feels, however, that the longitudinal grade adopted was about the maximum permissible and that the use of higher unit stresses in the floor system than in the main trusses is a variation from general practice.

(*h*) The resulting structure is very pleasing in appearance.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

CONSTRUCTION PLANT AND METHODS FOR ERECTING STEEL BRIDGES

Discussion

BY MESSRS. HOWARD P. TREADWAY, BENJAMIN WILDER GUPPY, R. A. VAN NESS, AND J. G. TRIPP

HOWARD P. TREADWAY,[3] M. AM. SOC. C. E. (by letter).[3a]—This paper is a distinct addition to the literature on the subject of construction plant and methods for erecting steel bridges. It is a comprehensive general statement of this interesting subject, and, as such, little can be added by way of discussion. It naturally leads, however, to a discussion of details entering into the various problems at hand and could very well be used as the opening statement of an extensive treatise on definite problems that have been solved.

It is quite probable that almost any steel bridge can be erected by several different methods of procedure and by using different types of erection equipment. Ordinarily, the designing engineer submits, to erecting contractors, plans showing a finished structure, and it is then the problem of the erecting engineer to determine the most economical plan for the field work. Usually, it is only the cheapest one that ever has any publicity, because under ordinary circumstances it is the cheapest procedure that warrants the awarding of the contract. Occasionally, there are conditions in which special service or responsibility of the contractor is considered, and a more elaborate scheme submitted by such a contractor is given the preference at a higher price.

The erecting engineer makes a careful study of the plans, the location, and local conditions, as set forth by Mr. Reichmann. It frequently happens that he has to re-design certain members of a proposed structure in order to erect it safely, because the designing engineer has overlooked the fact that during the process of erection his structure will have stresses in these members in excess of those occurring in the finished balanced structure.

Often the plant necessary to do the work in the easiest and most economical way is not available, or it may be too expensive to acquire, and

NOTE.—The paper by A. F. Reichmann, M. Am. Soc. C. E., was presented at the meeting of the Construction Division, New York, N. Y., January 16, 1930, and published in December, 1931, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[3] Pres. and Treas., Kansas City Bridge Co., Kansas City, Mo.

[3a] Received by the Secretary January 11, 1932.

the equipment that the erector has on hand has to be adapted to the problem. All these conditions having been considered a scheme is worked out, and an estimated cost is made, which bid is submitted to the customer.

If the field engineer develops a method that is safe under all conditions he is more than likely to be a high bidder. Therefore, it often happens that the field erector, in considering the flood conditions of a stream or the foundation material for the falsework, takes a long chance and offers to do the job based on ideal weather conditions. If these weather conditions prevail he is a "smart" man, and his performance is applauded. If, however, he is bordering on the unsafe side and conditions go against him, he possibly incurs great loss both to himself and to the customer.

These are a few of the general ideas that can be added to a paper such as that presented by Mr. Reichmann. A great many details pertaining to special jobs could well be discussed to illustrate the various examples cited by the author.

BENJAMIN WILDER GUPPY,[4] M. AM. SOC. C. E. (by letter).[4a]—The author mentions the requirement for the maintenance of traffic when replacing an old span as one factor that determines the method to be adopted for erecting the new one. In such a case it is necessary to remove the old span and sometimes alter and repair the existing substructure.

The type of superstructure to be removed and its condition and the repairs and alterations required to adapt the substructure to carry the new span and the heavier loads are factors that must also be considered in determining the erection method to be used. The writer recalls one case—the renewal of the superstructure of an important double-track railroad bridge over a wide river—where these were found to be the governing factors for determining the method to be used for the erection of the new superstructure and also for developing its design.

R. A. VAN NESS,[5] M. AM. SOC. C. E. (by letter).[5a]—The author has compiled a paper which condenses the subject in such form as to make it of real value as permanent information for the files of interested engineers, as well as educational matter for structural student engineers. To those who are closely in touch with bridge erection in one field or another, the paper revives a general appreciation of the entire subject.

The Company with which the writer is connected has maintained a steel bridge erection plant for many years. At present, it comprises rolling equipment to house a normal force of 30 persons; one derrick car capable of setting a 106-ft. Cooper E-65 girder ahead; one locomotive crane with a capacity of 75 tons; two gasoline-operated air compressors, capacity 310 cu. ft.; one two-unit, electric arc (300 amperes) welding machine; an oxy-acetylene torch; and miscellaneous small equipment.

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[4] Engr. of Structures, B. & M. R. R., Boston, Mass.

[4a] Received by the Secretary December 23, 1931

[5] Bridge Engr., A. T. & S. F. Ry. System, Chicago, Ill.

[5a] Received by the Secretary January 25, 1932.

Most of the Company's bridge steel is erected with this plant and no work is contracted except that which must be erected before Company forces would be available and other work of such magnitude that it requires a large outside plant for the most economical erection.

J. G. TRIPP,[6] M. AM. SOC. C. E. (by letter).[6a]—Only one addition is suggested for Mr. Reichmann's paper. The author does not stress sufficiently the fact that the solution of construction and erection problems should precede the design of a bridge. Mr. Reichmann follows this practice in his organization, but too often such considerations are passed by and left for the erector to make the best of poor "construction symmetry"—that ogre of the construction man. For example, it is not an uncommon occurrence in inaccessible territory to be faced with the fitting of the design to such a simple thing as the available plant of the hauling contractor in order to determine the sizes that can be handled into the location of the structure. Natural obstructions, road grades, etc., often cause complications difficult to overcome. Such structures as the Marble Canyon Bridge, in Arizona, is a case in point. The designers well knew that their problem was not so much stress, strain, and connections, but such simple things as maximum sizes of pieces to be transported from the railroad to the job, a matter of 120 miles over rough terrain. Design must accommodate erection methods and available plant to obtain economical results.

Mr. Reichmann's tabulation of erection conditions is particularly well stated. In the writer's opinion, he might have added:

"6.—Specification limitations:

(*a*) Eight-hour laws.

(*b*) Inspection routine."

If the specifications are inelastic and their effect unweighted in the estimate to erect, costly results and even defeat of the plans based entirely on the other five major conditions might follow.

As the author states, equipment and plant types are legion. The writer has found that little refinements on standard equipment pay big dividends. Simple devices enabling men to reach into difficult places; tool kits in basket slings; visual and manual signal assistance; proper power cord attachments, etc., are examples. Special refinements in cableway control apparatus and power take-ups on stay-lines pay handsomely.

In the design of structural steel falsework—another bridge problem—an important point is to build something that is easily dismantled without injuring the value of the dismantled parts. In this case, the study of erection methods and plant is paramount, and the design of the members is hedged in with limitations rarely found in permanent structures. In such cases all Mr. Reichmann's conditions must be evaluated for choice of method and plant, but final selections are tempered by considerations for easy and quick dismantling. For example, it might prove economical to erect a set of con-

[6] Care, Merritt-Chapman-Whitney Corporation, Rock Island, Ill.

[6a] Received by the Secretary February 6, 1932.

crete arch-bridge centering from a cableway, but it might also be very expensive to dismantle such centering with the same tool. The net result would be to develop some plan, such as trestle and locomotive cranes, or floating equipment and jacks, to displace the cableway. If the only consideration was to erect for permanence, the cableway might be chosen without further thought as being the most economical for the case in hand.

Mr. Reichmann's conclusion that there are no limits to the special cases of methods and equipment used in bridge erection, is fully seconded by the writer. If the designer, as well as the erector, will follow Mr. Reichmann's condition chart, few, if any, mistakes in judgment will result.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PUBLIC SUPERVISION OF DAMS
A SYMPOSIUM

Discussion

BY H. DE B. PARSONS, M. AM. SOC. C. E.

H. DE B. PARSONS,[8] M. AM. SOC. C. E. (by letter).[8a]—If there is to be State supervision of the design and construction of dams, it is the writer's opinion that the laws, enacted by the several States, should be uniform, or as nearly alike as may be possible. Engineers are the ones best qualified to furnish information for legislation that will be effective and not be burdensome. As the art of designing dams is in a progressive state, there should be no statutory limitations on design or method of construction. The writer does not favor a building code, as it probably would increase the cost of construction without providing the protection intended.

The following suggestion for supervision is offered for criticism. The basic idea is akin to arbitration, which has been proven fair, quick, and inexpensive, and has the approval of the Courts. The act should define the "owner" as the individual, corporation, or public body desiring to erect a dam, and the "State Engineer" (or a State bureau or department), as the representative of the general public; and should recite, in a clear manner, that approval does not convey any State guaranty of the safety of a dam, but that the public welfare has been safeguarded by bringing outside judgment to study the development.

The "owner" should apply to the State Engineer (or to the equivalent State bureau or department) for permission to construct. This application should be referred to a supervisory board composed of three commissioners, the first being appointed by the State Engineer, the second by the appellant (the engineers responsible for the design of the dam or development should not be eligible), and the third selected by the first two. The third commis-

NOTE.—The Symposium which includes the paper by A. H. Markwart, M. Am. Soc. C. E., presented at the meeting of the Power Division, New York, N. Y., January 16, 1930, and the paper by M. C. Hinderlider, M. Am. Soc. C. E., presented at the Technical Session, Sacramento, Calif., April 23, 1930, respectively, was published in January, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[8] Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.

[8a] Received by the Secretary January 22, 1932.

sioner should be the chairman and an engineer having no connection, direct or indirect, with contracting. Should a State be the "owner," then the State would be the appellant and should apply to the highest Court having jurisdiction, which Court should appoint the first commissioner. In case of disability, any vacancy should be filled by the appointment of another commissioner in accordance with the procedure outlined.

The "owner" should submit complete plans, with specifications and all his data regarding flood flows, rainfall, geologic formation, and borings. Additional data should be filed by the "owner" if requested by the board, provided such data are obtainable. The board should be empowered to retain a geologist, but must do so for all important or high dams, and for those in earthquake or volcanic localities. An important dam could be defined briefly as a dam located above and near enough to habitations or to cultivated or developed lands, that a complete failure would cause the risk of loss of life or heavy property damage. Similarly, a high dam could be defined as a dam, of any form, more than 100 ft. in height, above the bed of a stream or valley, which impounds, say, 6 000 acre-ft., or more. The decision of the board, in which two of the commissioners shall concur, should be made promptly, and a maximum time limit could be fixed by the act, with a proviso for an extension. The decision should be limited to either an approval or a rejection of the plan (including foundation) submitted. The commissioners' report should not make recommendations, but should simply state the reasons for approval or rejection.

The total cost of the board, including all fees, traveling expenses, and disbursements, should be approved by the State Comptroller, and then divided equally between the "owner" and the State. If the State is the "owner," then the entire cost should be paid by the State. If more than one State is affected, then the States should divide the cost *pro rata* as their interests may appear.

Either party should have the right to appeal from the decision of the supervisory board. The appeal should be to the highest Court having jurisdiction. This Court should hear arguments on all legal questions and render its decision, but should refer all technical questions of engineering, scientific data, or facts to an appeal board, composed of three commissioners to be appointed by the Court. The Court should select one from a list of at least two named by the "owner," and one from at least two named by the State Engineer. These two should submit a list of at least two (or more if asked by the Court) from which the Court should select the third commissioner. The act should fix the time in which this board should file its report, with a proviso for an extension; and also should provide for filling a vacancy through the disability of a commissioner. The appeal board should have power to retain its own geologist. The decision of this board, in which two of the commissioners shall concur, should be accepted by the Court as its decision on all questions of engineering and scientific fact. The cost of the appeal should be paid by the party losing the appeal.

State supervision for the construction of dams could be made a function of the State Engineer, or the equivalent bureau or department. All matters

in dispute could be referred to the supervisory board for settlement, with the right of appeal as described in the foregoing remarks. If deemed advisable, the "owner" could be made to file at the State Engineer's office, on completion of the work, a revised set of plans, with specifications, showing the dam as actually constructed.

The writer concurs with Mr. Markwart that there should be no exceptions because of ownership, except for unimportant structures the failure of which would not involve loss of life or property. Dams built by the departments and bureaus of the Federal Government, when erected on State lands, should not be exempted, if existing laws will so permit. All dams should be treated alike. Federal or State engineers are no more infallible, as Mr. Markwart states, than municipal, corporate, or private engineers.

The writer submits this suggestion for criticism and betterment. He believes, if State supervision is adopted, that legislation as briefly outlined herein would tend to secure a supervisory personnel that would be, as nearly as practicable, the equal in technical knowledge of those responsible for the design and construction of a great dam.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

SOCIETY AFFAIRS

ANNUAL REPORT OF THE BOARD OF DIRECTION FOR THE YEAR ENDING DECEMBER 31, 1931

In compliance with the Constitution, the Board of Direction presents its Report for the year ending December 31, 1931.

THE SEVENTY-NINTH YEAR

The outstanding general characteristic of the year 1931 is the extent of the business depression and its consequences upon every type of industrial and social endeavor. To what extent the Society has suffered is undoubtedly, therefore, a feature which should be fully set forth in this report, the object of which is to inform the membership of the Society's condition and its accomplishments. The reports of other years, as for 1929 or 1930, show activity in the initiation of new procedures or in the formulation of new policies. In this year, however, few new policies have been under consideration or few new procedures contemplated. The year has been one of consolidation of position rather than one of planning for further objectives. It closed with a better financial result than might be expected, a fact that is the more significant in the light of several circumstances.

Financial Condition

When adopted, a year ago, the budget reflected a more optimistic view than now may appear to be justified. It was recognized at that time that the new publication, *Civil Engineering,* could not be expected to become financially self-sustaining in its first year, and in order that the other activities of the Society should not be curtailed, a sum, to be drawn from previously accumulated funds, was placed in the budget as available for expenditure in 1931. By conservative administration, however, the year has closed with the withdrawal of only approximately $13 000 from accumulations of previous years.

Two or three other financial details may properly be brought into prominent relief. Such trust and other funds as the Society has in the form of securities have necessarily suffered in market value, but happily the total depreciation from purchase prices is less than 17% and the Society is under no particular stress to dispose of them. All the Society's property is invested, the major portion in such form as to return a fixed annual income applicable to current expenditure. The dues collected in 1931 averaged $16.53 per member, whereas the disbursements, the largest in the history of the Society, were equivalent to $27.15 per member. Forty per cent. of the Society's service to members, it will be seen, therefore, is carried on out of income arising from

other sources than dues, *i. e.,* largely from the sale of publications, advertisements in *Civil Engineering,* the rental of the 57th Street property, and the return from the investment in the building in which the Society is housed.

Membership

There has been an exceedingly gratifying net growth in membership. Net growth is necessarily a combination of gains and losses, the latter through death or resignation, or otherwise. Applications fell off approximately 22% from the high year, 1927, but the net growth this past year, nevertheless, is 531, making a total membership as of December 31 of 15 190. Two elements in this figure of net growth should not be allowed to pass unnoticed. On the one hand it is augmented by the lenient attitude of the Board of Direction in remitting the dues of certain members under unusual financial distress, who have faithfully supported the Society, in many cases, for twenty or even thirty years. It was the pleasure of the Board to remove from these loyal members the pressure of back dues which they were unable to pay. The other significant detail contributing to the sustained net growth of the Society was the continued interest shown by the young men just graduating from the engineering schools and immediately seeking admission as Juniors in the Society. To-day, the number of Juniors in the Society is 618% of the number 10 years ago. At the end of 1921 there were 458 Juniors; to-day, a decade later, there are 2 835 of these younger men in the Society.

Publications

As this is the first full year of *Civil Engineering,* the publications of the Society merit specific mention. Aside from several important psychological effects derived, the publication of *Civil Engineering* has provided for an increase of 17.5% in the volume of technical matter available to the membership. The contribution to technical literature in 1929, through the *Proceedings,* could be measured as 1 867 000 words. For the year 1930, a comparison would be misleading, but for this year the corresponding figure for *Proceedings, Civil Engineering,* Manuals and including Memoirs, is 2 193 000 words. The publications of the Society constitute the medium through which all its technical developments are made available to the widespread membership.

Employment Service

An outstanding but not encouraging feature of the year is the alteration which has taken place in the Employment Service which is operated in conjunction with the other Founder Societies. The Engineering Societies Employment Service is not in any sense a manufacturer of jobs. Rather is it a mart for the exchange of commodities, jobs on the one hand and services on the other, and in this year of curtailment on all sides the function of the Employment Service as an exchange has necessarily suffered. In 1929, its year of maximum influence, the Service made profitable contacts for 1 664 engineers. In 1931 this figure fell to 691, but it remains significant that even in its poorest month, December, it was able to direct 35 engineers to full paid positions.

The Function of the Staff

The year 1931, particularly, has intensified the significance of the function of the Society's paid staff. In many respects the Society may be likened to an industrial or manufacturing process in that through the medium of the staff the contributions of individual members are to be returned to the membership as a co-ordinated product. Members to the number of 1 073 contributed directly to the Society's administration. Members to the number of 1 478 contributed individually to its technical publications, and 738 members were engaged on its technical committees, a total of 3 289 individual contributing members. The function of the staff has been to fabricate these individual contributions through various processes into a harmonious whole.

The collection of dues and the payment of bills are bookkeeping operations readily visualized, the extent of which may perhaps be more clearly comprehended when it is realized that the business conducted in 1931 is almost exactly double that of the year 1921, another interesting comparison permissible with conditions existing only a decade ago. The process of election to membership and of transfer in grade is another operation made up of many individual contributions which, co-ordinated by the staff, eventuated in the election and transfer of 1 295 members. Similarly, it was the duty of the staff to administer the details incident to the dropping, resignation, and death of 524 members.

The preparation of the Year Book with something like 7 400 address or other changes yearly; the handling of 14 000 ballots; preparations for the mailing of 613 000 individual pieces of mail, or approximately 2 000 for each working day, are operations involving close application of especially qualified staff members.

The production of the Society's publications, however, and this year particularly, has been the process with the most exacting ramifications. *Transactions* was printed in New York. *Proceedings* is printed at Albany, N. Y. *Civil Engineering* is printed at Easton, Pa. The Year Book and Aims and Activities were printed at Baltimore, Md. Memoirs and Manuals were printed at Albany and in New York, and the special communications and many printed forms in use were produced upon competitive bid by local printers. Illustrations and diagrams are produced under four different processes at eleven different places.

To collate and to produce punctually in consistent style, through these agencies, the contributions of 1 478 members to the Society's publications is the duty of the Editorial Department. The continued growth of Society membership and the increase in the volume of Society publications has necessitated a staff now numbering 53, a figure nearly double that of eleven years ago, a third permissible comparison with conditions of a decade ago.

Engineering-Economics and Finance Division

One of the Society's activities properly to be recorded as a matter of progress, is the organization of the Engineering-Economics and Finance Division. This Division, authorized by the Board at its January meeting.

organized its Executive Committee at once. It now has 312 enrolled members, and provided a session at the recent meeting in St. Paul. It has two committees organized, one studying the economic values of bridge clearances and another the principles of valuation of engineering structures. This is the tenth of the Divisions formed to act as the Society's agencies for the intensive development of technical specialties.

Registration of Engineers

Properly also to be recorded as an adopted Society policy is the reply made to the request by American Engineering Council of its member-organizations, wherein the Society set forth that it favors the registration of engineers. The Society's Committee on Registration of Engineers continued its practice of calling together representatives of other interested engineering societies for the formulation of what is to be submitted as a model law for the registration of engineers wherever the adoption of such a law is contemplated.

Earthquake Lectures

Yet one other outstanding event remains to be recorded, namely, the visit to this country of Professor Kyoji Suyehiro, the eminent Japanese seismologist, Director of the Earthquake Research Institute of Tokyo Imperial University. At the suggestion and through the generous financial aid of John R. Freeman, Past-President and Hon. M. Am. Soc. C. E., Professor Suyehiro was invited to deliver a series of lectures on the application of engineering principles in the design of structures to resist earthquake shock. In October and November, 1931, he lectured before various Universities, Societies, and Clubs on both sides of the continent. Through a combination of Professor Suyehiro's generosity in relinquishing a considerable part of his honorarium, the continued generosity of Mr. Freeman, and the use of Society funds, these lectures will be printed by the Society.

MEETINGS OF THE BOARD OF DIRECTION

There have been five meetings of the Board of Direction during 1931:

January 19-20, New York, N. Y.
January 22, New York, N. Y.
April 13-14, Norfolk, Va.
July 6-7, Ranier National Park, Wash.
October 5-6, St. Paul, Minn.

There have been three meetings of the Executive Committee:

April 13, Norfolk, Va.
October 4, St. Paul, Minn.
December 14, New York, N. Y.

MEMBERSHIP

The changes in membership are shown in the following table:

	Jan. 1, 1931			Jan. 1, 1932			Losses				Additions			Totals		
	Resident	Non-Resident	Total	Resident	Non-Resident	Total	Transfer	Resignations	Dropped	Died	Transfer	Election	Reinstatement	Loss	Gain	Increase
Honorary Members..	3	14	17	5	13	18	0	0	0	3	*4	0	0	3	4	1
Members.....	1 036	4 778	5 814	1 022	4 848	5 870	4	48	27	98	†118	106	9	177	233	56
Associate Members...	1 078	5 106	6 184	1 081	5 256	6 337	118	64	88	30	‡134	314	5	300	453	153
Juniors.....	545	1 957	2 502	553	2 282	2 835	133	32	¶117	5	....	618	2	287	620	333
Affiliates.....	49	86	135	42	83	125	1	3	3	4	....	1	..	11	1	§10
Fellows.....	4	3	7	3	2	5	0	0	0	2	0	0	0	2	0	§2
Total.....	2 715	11 944	14 659	2 706	12 484	15 190	256	147	235	142	256	1 039	16	780	1 311	531

* 4 Members.
† 118 Associate Members.
‡ 133 Juniors, 1 Affiliate.
§ Decrease.
¶ 3 Juniors dropped on account of age limit.

New Members and Net Increase

The following table shows the new members and the net increase during the past ten years. The diagram on page 6 gives membership statistics for the same period.

	1922	1923	1924	1925	1926	1927	1928	1929	1930	1931
New Members*..	636	733	715	795	1 072	1 139	1 244	1 066	1 139	1 055
Net Increase....	246	320	262	111	721	755	820	508	574	531

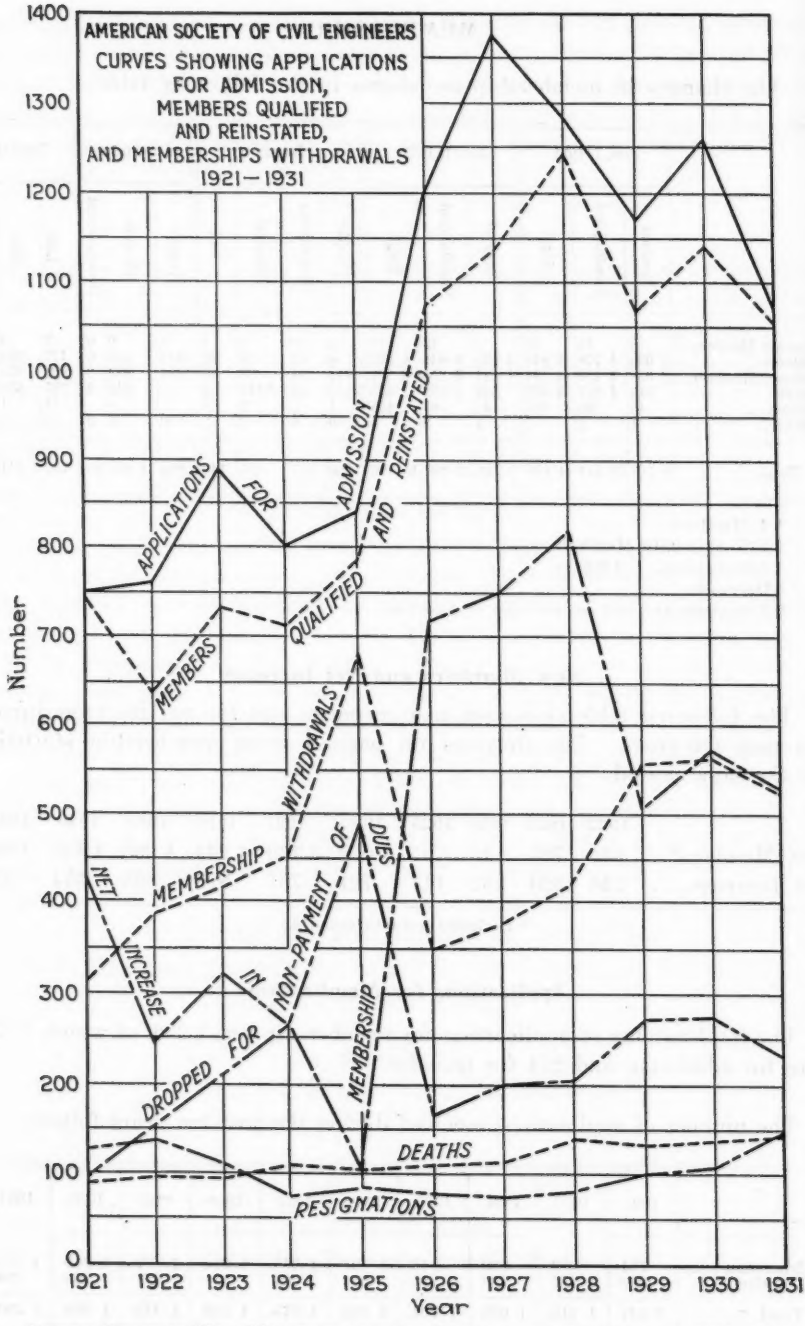
* Includes reinstatements.

Applications for Membership

The total number of applications for membership was 1 296, of which 1 072 were for admission and 224 for transfer.

The number of applications received during the past ten years follows:

	1922	1923	1924	1925	1926	1927	1928	1929	1930	1931
For Admission.....	761	896	800	848	1 194	1 374	1 284	1 172	1 260	1 072
For Transfer.....	260	217	239	256	292	304	274	271	338	224
Total.....	1 021	1 113	1 039	1 104	1 486	1 678	1 558	1 443	1 598	1 296



CURVES SHOWING NEW MEMBERS AND NET INCREASE IN MEMBERSHIP, 1921–1931.

DEATHS

The losses by death during the year number 142, and are as follows:

Past-Presidents (2):

George Fillmore Swain*
George Smedley Webster

Honorary Members (2):

Howard Adams Carson
Luigi Luiggi

Members (97):

Raymond Edmond Adams
William George Addis
Andrews Allen
Herbert Leander Aulis
Frank Harrison Bailey
John Goodin Bailhache
Samuel Pond Baird
Gustav Storm Bergendahl
James Wartelle Billingsley
Charles Abel Bowman
Alexander Gordon Brinckerhoff
Martin Joseph Caples
St. John Clarke
Robert Morris Clayton
Samuel Cecil Coddington
James Retzer Comly
Alger Adams Conger
Charles James Craigmile
Philip Herrick Dater
Frank Eugene Davidson
John Stanley Dawson
Harry Espenship Detwiler
Harold Mansfield Eddy
Charles Clement Elwell
Frank Collins Emerson
Lloyd Tilghman Emory
Howard Richards Farnsworth
Frank Foster
Edward Gagel
Horace Delphos Gates
Ernest Lee Griggs
Harry Hawgood
Eugene Christopher Heron
Harry Seymour Hodge
Edward Willard Howe
Edward Henry Hurry
Edgar Jadwin
George Portlock Janes
Clinton Hall Kearny
Alfred Lawrence Kehoe
Traugott Francis Keller
Sentaro Kondo
Olin Henry Landreth
William Henry Lawton
George Herbert Leland
John Lundie
John Derby Cornelius Mackey
Ernest McCullough
Maynard Sargent McDaniel
Anson Burlingame McGrew
Edwin Harrison McHenry
Marsden Manson
Horace Miller Marshall
Montgomery Meigs
Joseph Davis Metcalfe
Sheldon Elton Minor
John Mitchell Moncrieff
Richard Montfort
Daniel William Murphy
Ralph Hadlock Ober
Frederick Sylvester Odell
Pennell Churchman Painter
Arthur Pew
William Collins Phelps
Harry Longyear Preston
Samuel Moreau Purdy
Albert Reesor Raymer
Frederic Adams Reimer
Henry Clay Ripley
Knud Sophus Riser
William Lawrence Saunders
Philip Schuyler
Mitsugu Sengoku
Michael Jacobs Shamroy
Charles Rogers Shepley
Orville Campbell Skinner
Willis Appleford Slater
Jay L. Stannard
Eugene Washington Stern
Frank Stoddard Stevens
Carlton Strong
Henry Allen Symonds
Lewis Thornburg
Lester Waldo Tucker
Gustave R. Tuska
Harry Lee Van Zile
George William Volckman
Fred J. Wagner
Samuel Tobias Wagner
John Simpson Walker
Robert Smith Wallace
Andrew Jackson Wiley
Gardner Stewart Williams
Henry Dickinson Woods
Charles Worthington
Adolph Yappen
Elmer Zarbell

Associate Members (30):

Albert Ross Bailey
Newton Davis Benson
Morton Burden
Armand John Calleri
Clinton Hervey Crooks
Emil Frank Cykler
Robert Morris Drake
Preston Burt Easterbrooks
William Edward Fitzgerald
Robert Dickson Alison Frew
James Henry Gallivan
James Hendricks Hallett
Horace Malcolm Hastings
Lewis Stirling Hereford
Edgar Malin Hoopes, Jr.
Frank Burdette Kiel
John Thomas McLure
Kenneth Ames Marsh
Morris Meyers
Harry Christian Poske
William Couts Randolph, Jr.
Henry English Read
Homer Austin Reid
Charles Wallace Resmaw
Herbert Jones Sowden
Herbert Harris Starr
Edward Charles Strathmann
Samuel Houston Ware
Charles Edward Willacker
Ferd Clark Wintermute

Affiliates (4):

William Josiah Karner
Arthur James Moxham
Edward Alfred Simmons
William Greene Stone

Juniors (5):

Thomas Rogers Hughes
Elmer Barthold Lawson
Bernard Joseph Merickel
John Harry Miller
James Russell Vaughan

Fellows (2):

Edward Dean Adams
John Joseph Albright

*Also an Honorary Member.

ENGINEERING SOCIETIES LIBRARY

The statistics which follow give comparative figures for 1930 and 1931 of the Engineering Societies Library:

Additions:	1930		1931	
Volumes (by gift)	2 400		2 106	
" (by purchase)	1 327	3 727	1 304	3 410
Pamphlets (by gift)	3 043		3 894	
" (by purchase)	214	3 257	251	4 145
Maps (by gift)	258		198	
" (by purchase)	43	301	10	208
Searches		75		94
Total additions		7 360		7 857
Permanent collection		139 916		144 093
Expenditures for books, periodicals, binding, supplies, and salaries (approximate)		$48 000		$52 011.70
The Library was used by		40 000		42 659
Including personal visits by		25 113		27 943
Volumes catalogued		4 276		4 477
Cards added to catalog		27 264		29 596
Total catalog cards, arranged under subject		421 345		450 939
Searches made		160		117
Translations made		246		242
Total number of words		691 700		564 400
Photoprints made		46 425		41 614
Number of persons securing photoprints		4 679		4 585
Receipts for service		$21 560		$17 031.56
Members borrowing books		250		206

EMPLOYMENT SERVICE

The Employment Service has offices in New York, N. Y., Chicago, Ill., and San Francisco, Calif.

The number of men placed during 1931 has averaged about 58 per month. The following table shows the registrations and placements in the three offices:

Month	Men Registered				Men Placed			
	New York	Chicago	San Francisco	Total	New York	Chicago	San Francisco	Total
January	121	49	104	274	36	10	14	60
February	141	64	81	286	40	9	13	62
March	178	73	123	374	50	12	20	82
April	129	81	126	336	45	10	11	66
May	122	94	103	319	35	6	15	56
June	122	104	112	338	41	5	9	55
July	134	97	85	316	42	9	13	64
August	115	46	79	240	36	6	9	51
September	98	94	78	270	39	6	10	55
October	94	107	71	272	37	9	12	58
November	97	86	49	232	30	4	13	47
December	80	36	61	177	26	2	7	35
Total	1 431	931	1 072	3 434	457	88	146	691

PUBLICATIONS

One volume of *Transactions,* ten numbers of *Proceedings,* twelve numbers of *Civil Engineering,* a Year Book, Engineering Manual No. 7 ("Government Services Available to Civil Engineers"), and a new edition of Aims and Activities, have been published in 1931. Publication of Memoirs in *Proceedings* was discontinued with the September, 1930, number. Since that time approximately 170 memoirs have been published in pamphlet form, many of which are included in Vol. 95 (1931) of *Transactions.*

The stock of the various publications of the Society kept on hand for the convenience of members and others now amounts to 127 399 copies, the cost of which to the Society for paper and press work only has been $22 550.75.

The table (see page 10) shows the cost per page for text and illustrations in *Proceedings, Transactions,* and *Civil Engineering* for the past seventeen years.

The various topics developed in *Proceedings, Transactions,* and *Civil Engineering* during the year, and the number of pages devoted to each, follows:

Subject	*Transactions,* pages	*Proceedings,* pages	*Civil Engineering,* pages
City Planning	61	30	39
Contracts	....	....	18
Dams	185	186	34
Drainage and Irrigation	140	56	21
Engineering Economics	....	....	6
Engineering Education	....	....	6
Engineering History	....	....	28
Engineering Profession	9	....	13
Floods	....	....	15
Foundations	....	....	14
Handling of Materials	....	....	5
Highway Engineering	69	97	59
Hydrology, Hydraulics	110	120	32
Industrial Operations	....	....	5
Materials of Engineering	147	80	21
Mathematics	....	88	....
Oil Industry	....	....	6
Power Plants	18	....	15
Power Transmission	50	....	....
Railways	....	....	9
Rainfall	98	142	5
Rivers and Harbors	....	....	45
Sanitation	....	....	10
Sewage Disposal	21	20	23
Soils	....	80	6
Structural Engineering	162	446	86
Surveying	12	42	48
Transportation	90	18	13
Tunnels	45	94	16
Water Power	16	2	18
Waterways	111	36	15
Water-Works	74	54	18
Letters to the Editor	....	....	126
Memoirs	247	....	....
Society Affairs	....	26	71
Application for Admission and Transfer	....	318	....
Items of Interest	....	....	26
Local Sections	....	....	15
Student Chapters	....	....	7
Current Periodical Literature	....	....	36
Recent Books	....	....	5
Men and Positions Available	....	....	15
Changes in Membership Grades	....	....	19
News of Engineers	....	....	12
	1 665	1 935	981

TABLE SHOWING NUMBER AND COST OF PAGES AND COST OF ILLUSTRATIONS FOR *Transactions, Proceedings,* AND *Civil Engineering.*

Year	Issues	Edition	Pages: Per volume	Pages: Total	Issues	Edition	Pages: Per volume	Pages: Total	Total pages	Total cost	Cost per page	Cost	Per-centage of total cost	Cost per page
	TRANSACTIONS				*PROCEEDINGS* PART I				*PROCEEDINGS* (PART I)* and *TRANSACTIONS*			ILLUSTRATIONS		
1915	2	8 600	3 130	26 900 000	10	8 425	3 668	30 900 000	57 800 000	$47 934.16	$0.00083	$3 684.68	7.7	$0.000064
1916	1	8 400	2 301	19 330 000	10	8 350	2 892	24 140 000	43 470 000	35 645.65	0.00082	1 403.12	3.9	0.000032
1917	..	....	....	.....	10	8 550	3 432	29,350 000	29 350 000	28 608.18	0.00097	3 703.97	12.9	0.000126
1918	1	8 700	1 879	16 340 000	10	8 950	2 341	20 950 000	37 290 000	33 785.64	0.00091	1 192.20	3.5	0.000032
1919	1	9 000	1 775	15 980 000	8	9 100	2 096	19 075 000	35 055 000	32 082.69	0.00091	1 128.53	3.5	0.000032
1920	..	....	....	.....	10	10 142	2 014	20 440 000	20 440 000	23 446.34	0.00115	2 552.27	10.9	0.000125
1921	2	10 000 10 500	2 479 993	35 212 000	10	10 680	1 834	19 450 000	54 662 000	66 298.39	0.00121	2 034.72	3.1	0.000037
1922	1	10 900	1 826	19 900 000	10	11 100	2 740	30 400 000	50 300 000	56 200.00	0.00112	3 700.00	6.6	0.000073
1923	1	11 200	1 808	20 250 000	10	11 500	3 210	36 915 000	57 165 000	60 612.83	0.00106	4 809.88	7.9	0.000084
1924	1	11 500	1 515	17 440 000	10	11 750	2 612	30 700 000	48 140 000	47 573.72	0.00099	4 579.04	9.6	0.000095
1925	1	11 400	1 538	17 533 000	10	11 800	2 910	34 338 000	51 871 000	47 409.07	0.00091	6 359 19	13.4	0.000122
1926	..	12 100	1 786	21 610 000	10	12 200	3 046	37 161 000	58 771 000	53 473.54	0.00091	5 080.91	9.5	0.000087
1927	2	12 900 13 200	1 234 1 229	15 919 000 16 223 000	10	13 050	3 720	48 546 000	80 688 000	65 972.76	0.00080	9 279.49	14.0	0.000126
1928	1	14 000	1 860	26 040 000	10	14 200	4 016*	57 027 000	83 067 000	61 587.98	0.00074	7 249.00	12.4	0.000087
1929	1	14 500	2 030	29 435 000	10	14 540	3 566	51 850 000	81 285 000	58 388.39	0.00072	4 011.95	6.9	0.000049
1930	1	15 100	1 794	27 090 000	10	15 170	2 936	44 540 000	71 630 000	52 881.95	0.00074	4 189.56	7.9	0.000058
1931	1	15 500	1 720	26 660 000	10	15 820	1 968	31 133 000	57 793 000	42 342.33	0.00073	3 680.95	8.7	0.000064
	CIVIL ENGINEERING													
1930	....	.....	.....	.....	3	16 230	248	4 025 000	4 025 000	8 969.66	0.00223	995.06	10.1	0.000247
1931	....	.....	.....	.....	12	16 275	1 200	23 436 000	23 436 000	41 012.75	0.00175	6 271.40	15.3	0.000321

* Includes Part III, May 1928, 288 pp.

Summary of Publications for 1931

	Issues	Average edition	Total pages	Plates	Cuts
Proceedings (monthly numbers)	10	15 820	1 602	..	482
Civil Engineering (monthly numbers)	12	16 275	1 200	..	977
Transactions, Vol. 95	1	15 500	1 720	..	351
Year Book	1	15 500	506	1	3
Engineering Manual No. 7	1	16 000	36	..	6
Aims and Activities	1	23 500	38	1	5
Total	26	....	5 072	2	1 824

The cost of publications, as determined by the bills actually paid during the year, has been:

For *Proceedings:*		
Paper, Printing, Mailing	$19 855.66	
Paper, Printing, 4 060 Extra Copies of Papers, Reports, Discussions	590.33	
Binding, Envelopes	3 086.74	
Cuts	3 654.35	
Postage (Extra Copies)	1 237.09	
Traveling Expenses	71.44	
Copyright and Sundries	284.00	
Total		$28 779.61
For *Transactions:*		
Paper, Printing, Mailing	$13 475.20	
Extra Copies of Separate Papers (5 650)	952.97	
Binding,[1] Boxes	1 884.24	
Cuts (Corrections)	26.60	
Postage (Extra Copies)	493.34	
Copyright	4.10	
Total		$16 836.45
For *Civil Engineering:*		
Paper, Printing, Binding, Mailing	$33 754.92	
Separate Copies of Papers (57 200)	2 060.69	
Envelopes	911.99	
Cuts	6 271.40	
Technical Index	959.70	
Circulating and Advertising Department (Includes Traveling Expenses)	1 758.78	
Traveling Expenses (*Civil Engineering*)	496.04	
Postage (Extra Copies)	257.88	
Copyright and Sundries	74.44	
Total		$46 545.84

[1] Paper binding only.

For MEMOIRS (Pamphlets):		
Paper, Printing, etc., 12 900 copies		$ 1 330.75
For YEAR BOOK:		
Paper, Printing, Mailing	$6 391.01	
Envelopes	86.24	
Plate and Cuts	56.31	
Postage (Extra Copies)	1 358.98	
Total		$ 7 892.54
For AIMS AND ACTIVITIES:		
Paper	$ 336.62	
Printing, Binding	864.54	
Envelopes	41.69	
Cuts and Plate	29.43	
Postage (Extra Copies)	205.41	
Total		$ 1 477.69
For ENGINEERING MANUAL No. 7:		
Paper, Printing	$ 523.95	
Envelopes	80.70	
Cuts	162.60	
Postage	202.40	
Total		$ 969.65
For MISCELLANEOUS:		
Salaries, Technical Department	$48 975.26	
Extra Copies of 1930 Board of Direction Report (1 262)	196.22	
Cuts for Board of Direction Report	3.31	
Postage (Miscellaneous Publications)	234.51	
Copyright for Index to *Transactions*	2.18	
Total		$49 411.48
SUMMARY:		
For *Proceedings*	$ 28 779.61	
For *Transactions*	16 836.45	
For *Civil Engineering*	46 545.84	
For Memoir Pamphlets	1 330.75	
For Year Book	7 892.54	
For Aims and Activities	1 477.69	
For Engineering Manual No. 7	969.65	
For Miscellaneous	49 411.48	
Total	$153 244.01	
Deduct amount received from sale of publications and financial support	12 733.80	
Net expenditures for publications in 1931	$140 510.21	

READING ROOM OF THE SOCIETY

The attendance at the Reading Room during the year was 3 557.

Two hundred and seventy-four periodicals are regularly received. Included in this number are many foreign periodicals, also a number of literary magazines and several daily newspapers.

A number of new books were added to the Reading Room collection. Some of these were gifts from the authors and publishers.

MEETINGS

Six meetings of 8 sessions were held during the year as follows: At the Annual Meeting, at New York, N. Y., 1 (2 sessions); at the Spring Meeting, at Norfolk, Va., 1 (2 sessions); at the Annual Convention, at Tacoma, Wash., 1 (2 sessions); at the Fall Meeting, at St. Paul, Minn., 1 (2 sessions); and 2 regular meetings of the Society held in the Engineering Societies Building, New York, N. Y.

At the Quarterly Meetings of the Society, there were presented 17 formal papers, many of which were illustrated with lantern slides, 1 Symposium, and 14 Addresses. Many of these papers have been published in *Civil Engineering,* as noted subsequently. Student Chapter and Local Section Conferences were held at Norfolk, Va., Tacoma, Wash., and St. Paul, Minn.

During the year Specifications for Boundary Surveys prepared by the Committee of the Surveying and Mapping Division on Boundary Surveys, and a Progress Report of the Sanitary Engineering Division on New Jersey Sewage Experiments (presented at the Spring Meeting at Norfolk, Va.), were published in May and August, 1931, *Proceedings,* respectively. There were also published in *Proceedings* 28 papers, 1. Symposium, and 1 Bibliography.

The number of members and others who took part in the preparation and discussion of these papers, discussions, specifications, reports, symposium, and bibliography, published in *Proceedings* was 116, and 141 discussions on papers published in 1930 *Proceedings,* were also included in *Proceedings* for 1931.

The total attendance at the meetings of the Society was approximately 3 150. The registered attendance at the Annual Meeting was, 1 780; at the Spring Meeting, 452; at the Annual Convention, 511; and at the Fall Meeting, 400.

The dates of the meetings of the Society during the year, together with the titles of papers (abstracts), addresses, etc., presented thereat, are as follows:

January 21, 1931,[1] "Government Day": A Symposium on the Work of the Federal Bureaus as Affecting Civil Engineers.

April 15, 1931 (Two Sessions)[2], "The Port of Hampton Roads" by the Hon. J. Gordon Bohannan; "Mississippi River Flood Control and Channel Maintenance", by Lytle Brown, M. Am. Soc. C. E.; "Transportation Problems of

[1] *Civil Engineering,* March, 1931, p. 489, *et seq.*

[2] *Loc. cit.,* June, 1931, p. 791, *et seq.*

the Railroads", by Elisha Lee, Esq.; "Coast Erosion and the Remedies", by E. J. Dent, M. Am. Soc. C. E.; "Aerial Commercial Transportation", by V. P. Conroy, Esq.; "Army Aviation", by Captain George C. Kenney, Air Corps, U. S. A.; and "Naval Aviation" by Captain Kenneth Whitney, U. S. N.

July 8, 1931 (Two Sessions)[3], "The Engineer's Growing Civic Responsibilities",[4] Address by Francis Lee Stuart, President, Am. Soc. C. E.; "Developing the Pacific Northwest", by Ralph Budd, M. Am. Soc. C. E.; "The Columbia River—For Irrigation and Power", by John S. Butler, M. Am. Soc. C. E.; and "Irrigation—A National Problem", by R. K. Tiffany, M. Am. Soc. C. E.

October 7, 1931 (Two Sessions)[5], "The Trend of Mining and Concentrating Practice on the Mesabi Iron Range", by Russell H. Bennett, Esq.; "Economic and Political Phases of the Iron Mining Industry", by E. W. Davis, Esq.; "St. Paul's Development Program", by William N. Carey, M. Am. Soc. C. E.; "Economic Phases of Railroading in the Northwest", by W. L. Darling, M. Am. Soc. C. E.; and "The Upper Mississippi Waterways", by Hibbert M. Hill, Assoc. M. Am. Soc. C. E.

The papers, etc., published in *Proceedings* during the year were as follows:

January, 1931, "Design of a Reinforced Concrete Skew Arch", Bernard L. Weiner, Assoc M. Am. Soc. C. E.; "A Discharge Diagram for Uniform Flow in Open Channels", by Murray Blanchard, M. Am. Soc. C. E.; and "Legal Problems Involved in Establishing Set-Back Lines", by Clifton Williams, Esq.

February, 1931, "The Eight-Mile Cascade Tunnel, Great Northern Railway": A Symposium; "Measuring Irrigation Deliveries in the Panjab", by E. S. Lindley, M. Am. Soc. C. E.; and "Surveys for Swift River Reservoir of the Boston Metropolitan Water Supply", by N. LeRoy Hammond, M. Am. Soc. C. E.

March, 1931, "Flow of Ground Water as Applied to Drainage Wells", by M. R. Lewis, M. Am. Soc. C. E.; and "Gravity Dams Arched Down Stream", by B. J. Lambert, M. Am. Soc. C. E.

April, 1931, "An Analysis of Multiple-Skew Arches on Elastic Piers", by J. Charles Rathbun, M. Am. Soc. C. E.; and "Run-Off—Rational Run-Off Formulas", by R. L. Gregory and C. E. Arnold, Assoc. Members, Am. Soc. C. E.

May, 1931, "Manufacturing Concrete of Uniform Quality", by William M. Hall, M. Am. Soc. C. E.

August, 1931, "Financing Street and Highway Improvements", by R. W. Crum, M. Am. Soc. C. E.; and Bibliography of Physical Properties and Bearing Value of Soils, by Morris Schrero, Esq.

September, 1931, "Design of Large Pipe Lines", by Herman Schorer, Assoc. M. Am. Soc. C. E.; "Highway Construction Management", by T. Warren Allen, M. Am. Soc. C. E.; and "Construction Management", by Richard Hopkins, Esq.

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[3] *Civil Engineering*, September, 1931, p. 1075, *et seq.*
[4] *Transactions*, Am. Soc. C. E., Vol. 95 (1931), p. 1294.
[5] *Civil Engineering*, December, 1931, p. 1334, *et seq.*

October, 1931, "Soil Mechanics Research", by Glennon Gilboy, Jun. Am. Soc. C. E.; "Engineering Features of the Illinois Waterway", by Walter M. Smith, M. Am. Soc. C. E.; and "Construction Work on a Federal Reclamation Project", by Morris Mason, Jun. Am. Soc. C. E.

November, 1931, "Water Pressures on Dams During Earthquakes", by H. M. Westergaard, M. Am. Soc. C. E.; "Western Highway Practice, with Special Reference to California", by C. S. Pope, M. Am. Soc. C. E.; and "Low-Cost Bituminous Roads", by Walter N. Frickstad, M. Am. Soc. C. E.

December, 1931, "Effects of Bending Wire Rope", by Frederick C. Carstarphen, M. Am. Soc. C. E.; "Lake Champlain Bridge", by Charles M. Spofford, M. Am. Soc. C. E.; "Construction Plant and Methods for Erecting Steel Bridges", by A. F. Reichmann, M. Am. Soc. C. E.; and "Fulton Street, East River Tunnels, New York, N. Y.", by Miles I. Killmer, M. Am. Soc. C. E.

MEDALS, PRIZES, AND AWARDS

The award of Medals and Prizes for the year ending July, 1931, was as follows:

The Norman Medal to Floyd A. Nagler and Albion Davis, Members, Am. Soc. C. E., for their paper entitled "Experiments on Discharge Over Spillways and Models, Keokuk Dam".

The J. James R. Croes Medal to John R. Freeman, Past-President and Hon. M. Am. Soc. C. E., for his paper entitled "Flood Control on the River Po in Italy".

The Thomas Fitch Rowland Prize to Samuel A. Greeley, M. Am. Soc. C. E., and William D. Hatfield, Assoc. M. Am. Soc. C. E., for their paper entitled "The Sewage Disposal Works of Decatur, Illinois".

The Arthur M. Wellington Prize to George F. Schlesinger, M. Am. Soc. C. E., for his paper entitled "The Practical Utility of Highway Transport Surveys".

The Freeman Traveling Scholarship for the year 1931 was re-awarded to Hans Kramer, Assoc. M. Am. Soc. C. E.

The first award of the Alfred Noble Prize was made to Corbin T. Eddy, Esq., for his paper entitled "Arsenic Elimination in the Reverberatory Refining of Native Copper" published in the *Transactions* of the American Institute of Mining and Metallurgical Engineers, General Volume, 1931.

The Rudolph Hering Medal of the Sanitary Engineering Division was awarded to Samuel A. Greeley, M. Am. Soc. C. E., and William D. Hatfield, Assoc. M. Am. Soc. C. E., for their paper entitled "The Sewage Disposal Works of Decatur, Illinois".

LOCAL SECTIONS

There are at present 53 Local Sections, the Alabama Section and the Panama Section, approved by the Board January 19, 1931, having been added during the year. The name of the New York Section was changed to Metropolitan Section.

TECHNICAL DIVISIONS

During the year there were twenty Division Programs, of which two were double sessions, and two were joint sessions of two Divisions.

The Engineering-Economics and Finance Division, organized at the 1931 Annual Meeting, held its first session at the 1931 Fall Meeting in St. Paul, Minn.

These Technical Sessions were well attended and the policy of issuing instructions to authors and discussers respecting presentation of papers has been continued with favorable response on the part of speakers.

The policy of publishing abstracts in *Civil Engineering* of papers, discussions and reports presented at these sessions has been continued during 1931. March, June, September, and December, 1931, numbers of *Civil Engineering* were the particular media for these abstracts.

City Planning Division

January 22, 1931,[1] "The Civil Engineer's Part in the City Plan", by Morris Knowles, M. Am. Soc. C. E.; and "More Proof That City Planning Pays," by U. S. Grant, 3d, Assoc. M. Am. Soc. C. E.

July 9, 1931 (Joint Session with Highway Division),[2] Preliminary Report of Committee on Street Thoroughfares Manual; "Factors in Successful Highway Operation: Population Density Related to Roadway Area", by W. J. Fox, Assoc. M. Am. Soc. C. E.; "By-Passing Traffic Around Small Towns", by E. E. East, Assoc. M. Am. Soc. C. E.; "Increasing Highway Efficiency", by L. I. Hewes, M. Am. Soc. C. E.; and "Some City Planning Problems of Tacoma", by C. E. Putnam, M. Am. Soc. C. E.

October 9, 1931,[3] "Carrying on Regional Planning in the Region of Chicago, Ill.", by Robert Kingery, Esq.; and "Planning of St. Paul, Minn.", by George H. Herrold, M. Am. Soc. C. E.

Construction Division

January 22, 1931,[4] "Longest Continuous Railroad Trusses Yet Constructed", by Wilson T. Ballard, M. Am. Soc. C. E.; and "Improvements in Handling Building Material", by G. G. Wheat, Esq.

Engineering-Economics and Finance Division

October 9, 1931,[5] "The Controversy as to Clearances Over Navigable Waterways", by John F. Coleman, Past-President, Am. Soc. C. E.; and "The New Competition and New Horizons in Engineering", by Frederick H. McDonald, M. Am. Soc. C. E.

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[1] *Civil Engineering*, March, 1931, pp. 507, 524.
[2] *Loc. cit.*, September, 1931, p. 1104, *et seq.*
[3] *Loc. cit.*, December, 1931, p. 1327, *et seq.*
[4] *Loc. cit.*, March, 1931, pp. 536, 484.
[5] *Loc. cit.*, December, 1931, p. 1361, *et seq.*

Highway Division

January 22, 1931,[6] "A Summary of the Present State of Prequalification", by C. J. Tilden, M. Am. Soc. C. E.; and "Financing Street and Highway Improvements", by R. W. Crum, M. Am. Soc. C. E.[7]

April 16, 1931,[8] "Convict Labor in Highway Construction", by Henry G. Shirley, M. Am. Soc. C. E.; "Arbitration on Public Works Contracts: Advantages to Be Gained", by Charles H. Stevens, M. Am. Soc. C. E.; and "How Arbitration Works", by Philip A. Beatty, M. Am. Soc. C. E.

October 8, 1931,[9] "Highway Parasites", by C. M. Babcock, Esq.; "Highway Location and Design", by O. L. Kipp, Esq.; and "Highway Patrol in Minnesota", by Earle Brown, Esq.

Irrigation Division

July 9, 1931,[10] "Treated Lumber for Irrigation Structures", by L. Murray Grant, M. Am. Soc. C. E.; "Models Predict Flow Through Structures", by J. C. Stevens, M. Am. Soc. C. E.; and "Improving the Accuracy of Instruments", by L. Standish Hall, Assoc. M. Am. Soc. C. E.

Power Division

January 22, 1931,[11] "Natural Gas for Steam Power", by George I. Rhodes, M. Am. Soc. C. E.; "Record Breaking Hydro-Electric Turbines Installed", by John P. Hogan, M. Am. Soc. C. E.; and "Preventing Erosion Below Overflow Dams", by R. F. Ewald, M. Am. Soc. C. E.

July 9, 1931 (Joint Session with Construction Division),[12] "The Rock Island Development—Columbia River", by William D. Shannon, M. Am. Soc. C. E.; "Ariel Dam on the Lewis River", by Lyman Griswold, M. Am. Soc. C. E.; "Skagit River Developed by Seattle", by T. H. Carver, M. Am. Soc. C. E.; "Tacoma Completes Second Cushman Plant", by Verne Gongwer, M. Am. Soc. C. E.; and "Unusual Flow Phenomena", by Fred C. Scobey, M. Am. Soc. C. E.

Sanitary Engineering Division

April 16, 1931,[13] "Norfolk's First Essentials for Sanitation: Water Supply and Sewerage", by Walter H. Taylor, 3d, M. Am. Soc. C. E.; "Developing the Water Supply of Norfolk", by J. N. Chester, M. Am. Soc. C. E.; and Report of Committee on New Jersey Sewage Experiments.

October 8, 1931,[14] "The Sewage Disposal Situation at Minneapolis and St. Paul", by J. A. Childs, M. Am. Soc. C. E.; "The Grand Rapids Sewage Treatment Plant", by Milton P. Adams, Esq.; "Trickling Filter Loadings and Oxygen Demand Reductions", by J. A. Childs, M. Am. Soc. C. E, and George

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[6] *Civil Engineering*, March, 1931, p. 504.
[7] *Proceedings*, Am. Soc. C. E., August, 1931, p. 853.
[8] *Civil Engineering*, June, 1931, pp. 812, 847, 849.
[9] *Loc. cit.*, December, 1931, p. 1378, *et seq.*
[10] *Loc. cit.*, September, 1931, p. 1090, *et seq.*
[11] *Loc. cit.*, March, 1931, p. 513, *et seq.*
[12] *Loc. cit.*, September, 1931, p. 1101, *et seq.*
[13] *Loc. cit.*, June, 1931, p. 853, *et seq.*
[14] *Loc. cit.*, December, 1931, p. 1334, *et seq.*

J. Schroepfer, Jun. Am. Soc. C. E.; and Report of Committee on Filtering Materials.

Structural Division

January 22, 1931,[15] "Wind Bracing in Tall Steel Buildings", by C. R. Young, M. Am. Soc. C. E.; "New York's Tallest Skyscraper", by H. G. Balcom, M. Am. Soc. C. E.; and "Bank of Manhattan Building", by H. V. Spurr, M. Am. Soc. C. E.

July 9, 1931,[16] "The Lake Union Bridge Completed", by T. G. McCrory, M. Am. Soc. C. E.; and "Rope Strands for Long Suspension Span", by D. B. Steinman, M. Am. Soc. C. E.

Surveying and Mapping Division

April 16, 1931,[17] "The Utility of Geodetic Control Surveys", by R. S. Patton, M. Am. Soc. C. E.; "Surveys for River Studies", by Gerard H. Matthes, M. Am. Soc. C. E.; and "Aerial-Photography for Railroad Location", by M. A. Butler, M. Am. Soc. C. E.

October 8, 1931,[18] "The Recovery and Identification of Original United States Government Section and Quarter Section Corners", by J. S. Dodds, M. Am. Soc. C. E.; and "Surveys of the Upper Mississippi", by Lenvik Ylvisaker, Esq.

Waterways Division

April 16, 1931,[19] "Chesapeake Bay Port Developments: The Port of Baltimore", by Lt. Col. W. T. Hannum, Corps of Engineers, U. S. A.; "The Waterways of the National Capital", by Maj. Joseph D. Arthur, Jr., Corps of Engineers, U. S. A.; and "The Atlantic Intracoastal Waterway", by Gordon R. Young, M. Am. Soc. C. E.

MEMBERSHIP OF TECHNICAL DIVISIONS

There are now 14 432 members enrolled in the Technical Divisions as follows:

City Planning	1 556
Construction	2 591
Engineering-Economics and Finance	308
Highway	2 132
Irrigation	916
Power	857
Sanitary Engineering	1 626
Structural	2 761
Surveying and Mapping	836
Waterways	849

[15] *Civil Engineering*, March, 1931, p. 467, *et seq.*
[16] *Loc. cit.*, September, 1931, p. 1087, *et seq.*
[17] *Loc. cit.*, June, 1931, p. 836, *et seq.*
[18] *Loc. cit.*, December, 1931, p. 1357, *et seq.*
[19] *Loc. cit.*, June, 1931, p. 795, *et seq.*

STUDENT CHAPTERS

There are at present 99 Student Chapters. The following were organized during 1931:

Harvard Engineering Society (Harvard University)
Michigan College of Mining and Technology
Newark College of Engineering
Pennsylvania Military College

LANTERN LECTURES

The series of lantern lectures sponsored by the Committee on Student Chapters are being used for the third scholastic year, and the interest in this service is still growing.

The total number of meetings at which the lectures were used during the calendar year of 1931 is one hundred and eighty-five. Thirteen titles are now available, with others in prospect when the funds warrant the preparation of the material. By making up a duplicate set of each title it has been possible to show some of the subjects as often as four times in one month, at different colleges.

FINANCES

A payment of $15 000 was made on the mortgage.

The reports of the Secretary and Treasurer are appended.

By order of the Board of Direction,

GEORGE T. SEABURY,
Secretary.

January 18, 1931.

REPORT OF THE SECRETARY FOR THE

TO THE BOARD OF DIRECTION OF THE

GENTLEMEN:—I have the honor to present a statement of Receipts and
There is also appended a general Balance Sheet showing the condition of the

RECEIPTS

Balance on hand January 1, 1931			$38 468.89*
Entrance Fees		$ 19 800.00	
Current Dues		177 376.16	
Past Dues		11 949.46	
Advance Dues		54 425.99	
Binding for Members		8 642.00	
Badges		5 778.50	
Certificates of Membership		667.50	
Sale of Publications		11 228.80	
Interest on Deposits		398.17	
Interest on Investments		2 743.49	
Interest Accrued		420.14	
Annual Meeting		3 974.50	
Postage		392.83	
Advertising		30 666.30	
Miscellaneous		2 117.35	
Income from 57th St. Property:			
Credited to General Receipts	$42 000.00		
" " 57th St. Property Fund.	10 500.00	52 500.00	
Interest on Invested Funds		2 174.03	
From Engineering Foundation in Credit to:			
Special Committee on Earths and Foundations.		2 500.00	
Special Committee on Steel Column Research.		2 000.00	
Special Committee on Concrete and Reinforced Concrete Arches		1 500.00	
Special Publications		1 505.00	
Loan from Chase National Bank	$20 000.00		
Less Discount	150.00	19 850.00	
The Freeman Fund:			
Income and Expense		1 673.40	
Principal		132.63	
The Alfred Noble Fund:			
Interest on Invested Funds		770.00	
Interest on not Released Funds		4.42	
Rudolph Hering Medal Fund: Interest		21.46	
Merritt H. Smith Memorial: Interest		41.94	
City Planning Division:			
Dues		45.00	
Interest		17.87	
Surveying and Mapping Division: Dues		20.00	
Power Division: Interest		78.49	
Contribution to:			
Special Committee on Earths and Foundations.		925 00	
Sale of Securities		15 170.09	431 510.52
			$469 979.41

* For itemized statement, see p. 22.

YEAR ENDING DECEMBER 31, 1931

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Disbursements for the fiscal year of the Society, ending December 31, 1931. affairs of the Society.

Respectfully submitted,
GEORGE T. SEABURY,
Secretary.

DISBURSEMENTS

Salaries of Officers		$22 325.00
Retirement Allowances		6 954.48
Clerical Help		72 178.93
Traveling Allowance of Officers		20 136.88
Rent		14 214.00
Telephone		2 241.56
General Publications		9 569.76
General Printing		2 453.79
Postage		4 232.00
Binding		5 017.91
Badges		4 232.78
Certificates		441.60
Annual Prizes		345.81
Office Supplies		4 830.48
Furniture—Office Equipment		2 345.52
Current Business		4 231.12
Interest on Mortgage		1 875.00
Insurance		259.35
Reading Room		480.25
Miscellaneous		1 537.25
Employment Service		2 041.83
Library		9 342.48
American Standards Association		530.50
Local Sections		13 697.00
Technical Publications		143 674.25
Meetings		13 845.92
Technical Divisions		8 826.29
Technical Committees		14 925.77
Administrative Committees		1 544.24
Professional Committees		2 217.23
American Engineering Council		14 660.00
Payment on Mortgage		15 000 00
The Freeman Fund:		
Income and Expense		1 350.00
Principal		100.00
The Fowler Fund:		
Income and Expense		130.52
The Alfred Noble Fund:		
Income and Expense		1.20
Merritt H. Smith Memorial:		
Income and Expense		25.00
Rudolph Hering Medal Fund:		
Income and Expense		23.40
Loan from Chase National Bank	$19 850.00	
Discount	150.00	20 000.00
		$441 839.10
Cash on hand December 31, 1931		28 140.31*
		$469 979.41

* For itemized statement, see p. 22.

Report of Secretary (*Continued*)

ITEMIZED STATEMENT OF CASH ON HAND JANUARY 1, 1931

Society Funds in Chase National Bank, 23d Street.	$24 512.80	
" " " " " " 41st Street.	500.00	
Petty Cash (in hands of Secretary)	5 000.00	$30 012.80
57th Street Property Fund		632.14
Power Division	$2 078.01	
City Planning Division	435.90	
Surveying and Mapping Division	55.00	2 568.91
Special Committee on Stresses in Railroad Tracks.	$360.28	
Special Committee on Earths and Foundations (special account)	1 300.00	1 660.28
The Freeman Fund:		
Principal	$34.57	
Income and Expense	314.68	349.25
In Escrow, E. W. L.		393.02
The Alfred Noble Fund:		
Uninvested	$52.24	
Not Released	55.00	
Income and Expense	808.25	915.49
Phebe Hobson Fowler Fund		130.52
Merritt H. Smith Memorial Fund		1 103.34
Rudolph Hering Medal		497.11
Library and Prize Fund		206.03
		$38 468.89

ITEMIZED STATEMENT OF CASH ON HAND DECEMBER 31, 1931

Society Funds		$4 640.93
57th Street Property Fund		13 306.17
Power Division		2 156.50
City Planning Division		493.77
Surveying and Mapping Division		75 00
Special Committee on Stresses in Railroad Track.	$360.28	
Special Committee on Earths and Foundations	2 424.17	2 784.45
The Freeman Fund:		
Income and Expense	$638.08	
Principal	67.20	705.28
In Escrow, E. W. L.		393.02
The Alfred Noble Fund:		
Uninvested	$127.24	
Not Released	55.00	
Income and Expense	1 581.47	1 763.71
Merritt H. Smith Memorial Fund:		
Principal	$1 076.50	
Income and Expense	43.78	1 120.28
Rudolph Hering Medal Fund:		
Principal	$455.97	
Income and Expense	39.20	495.17
Library and Prize Fund		206.03
		$28 140.31

REPORT OF THE TREASURER OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS FOR THE YEAR ENDING DECEMBER 31, 1931

In compliance with the provisions of the Constitution, I have the honor to present the following report:

Cash on hand January 1, 1931		$ 38 468.89
Receipts		
From Current Sources January 1 to December 31	$353 423.54	
Rent from 57th Street Property	52 500.00	
From Engineering Foundation for:		
Special Committee on Concrete and Reinforced Concrete Arches	1 500.00	
Special Committee on Steel Column Research	2 000.00	
Special Committee on Earths and Foundations	2 500.00	
The Freeman Fund Investments	1 673.40	
Interest on Temporary Investments	2 743.49	
From Sale of Securities	15 170.09	431 510.52
Disbursements		
Payment of bills by audited Vouchers for Current Business, January 1 to December 31	$426 839.10	
Payment of Mortgage	15 000.00	
Cash on hand, December 31, 1931	28 140.31	
	$469 979.41	$469 979.41

Respectfully submitted,

OTIS E. HOVEY,

Treasurer.

BALANCE SHEET

ACCOMPANYING THE REPORT

ASSETS

Current Assets:			
Cash:			
In banks and on hand	$4 640.93		
On deposit with U. S. Post Office	200.00	$4 840.93	
Investment in City of Chicago tax notes, at cost (due September 15, 1930)		10 062.98	
Accounts receivable:			
Members, in arrears	$34 047.97		
Non-members	3 110.31	37 158.28	
Accrued interest on general investments		1 913.32	
Inventory of publications on hand		22 550.75	
Prepaid insurance premiums		303.00	
		$76 829.26	
Real Estate:			
Interest in real estate and other assets of United Engineering Trustees, Inc., exclusive of trust funds	$493 352.60		
218-220 West 57th Street, New York, N. Y., at estimated value, less depreciation	612 160.64	1 105 513.24	
Equipment:			
Furniture and office equipment	$77 430.44		
Less, reserve for depreciation	73 628.75	3 801.69	
Library:			
Cash expended for books, etc.	$22 122.22		
Donations (estimated)	72 310.83	94 433.05	$1 280 577.24
Fund Investments:			
Library and Prize Funds, etc.:			
U. S. Liberty, New York City, and R. R. bonds, at cost	$38 684.75		
Uninvested cash	206.03	$38 890.78	
The Fifty-seventh Street Property Fund:			
Securities, at cost	$49 079.24		
Uninvested cash	13 306.17		
Accrued interest on investments	748.43	63 133.84	
The Freeman Fund:			
Securities, at cost	$26 018.93		
Uninvested cash	67.20	26 086.13	
Alfred Noble Fund:			
Securities, at cost	$15 416.25		
Unexpended cash	127.24		
Accrued interest on investment	252.50	15 795.99	
Merritt Haviland Smith Memorial Fund		1 076.50	
Rudolph Hering Medal Fund		455.97	145 439.21
Cash in banks, representing unexpended balances of cash received for special purposes			8 260.27
			$1 434 276.72

We have examined the accounts of the AMERICAN SOCIETY OF CIVIL ENGINEERS inventories at cost, and the reasonableness of the estimates of property and library the Society as at that date.

NEW YORK, January 13, 1932.

DECEMBER 31, 1931

OF THE SECRETARY

LIABILITIES AND FUNDS

Mortgage payable, due February 1, 1932....	$30 000.00		
Interest accrued on mortgage.....	625.00		
1932 membership dues paid in advance.....	54 425.99		
Other membership credits, etc.....	2 431.16	$ 87 482.15	
Current fund surplus, including amount arising from revaluation of real estate.....		1 193 095.09	$1 280 577.24
Funds:			
Library and Prize Funds:			
Herbert Stewart Library Fund.....	$ 2 000.00		
Joseph G. Swift Library Fund.....	1 000.00		
Compounded Dues Fund	15 480.00		
Norman Medal Fund	1 000.00		
Rowland Prize Fund	1 222.50		
Collingwood Prize Fund	1 000 00		
Arthur M. Wellington Prize Fund.....	2 150.00		
Fellowship Fund	13 038.28		
Hiram F. Mills Legacy	2 000.00	$38 890.78	
The Fifty-seventh Street Property Fund.....		63 133.84	
The Freeman Fund		26 086.13	
Alfred Noble Fund	$15 543.49		
Interest accrued but not yet received on securities	252.50	15 795.99	
Merritt Haviland Smith Memorial Fund.....		1 076.50	
Rudolph Hering Medal Fund.....		455.97	145 439.21
Unexpended balances of cash received for special purposes:			
Special Committee on Stresses in Railroad Track...		$ 360.28	
Special Committee on Earths and Foundations.....		2 424.17	
Power Division		2 156.50	
City Planning Division.....		493.77	
Surveying and Mapping Division.....		75.00	
Rudolph Hering Medal Fund.....		39.20	
Freeman Fund income		638.08	
Alfred Noble Fund		1 636.47	
Merritt Haviland Smith Memorial Fund.....		43.78	
E. W. Lane (in escrow)		393.02	8 260.27
			$1 434 276.72

as at December 31, 1931, and (based upon the valuation of securities and publication valuations), in our opinion, the above balance sheet sets forth the financial condition of

LYBRAND, ROSS BROS. & MONTGOMERY,

Accountants and Auditors.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer*, and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that errors in the record be pointed out and a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from March 15, 1932.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

LIST OF APPLICANTS.

Names and Addresses of Applicants for Admission and for Transfer on this List. Arranged Alphabetically.

Name.	Address.	Page.
ABELL, DEWITT S.	Beacon, N. Y.	53
ALLISON, WILLIAM N.	Seattle, Wash.	53
BALABAN, EDWARD	New York City	53
BARRABEE, ARTHUR S.	New York City	54
BARTLEY, ROBERT J.	Bay Shore, N. Y.	54
BEHLEY, RUDOLPH J.	Milford, Conn.	54
BERSIG, WALTER C.	Brooklyn, N. Y.	54
BILLING, JOHN M.	Singapore, S. S.	55
BRODKIN, NATHAN D.	New York City	55
BRUNER, EDWIN L.	Los Angeles, Cal.	67
BURN, PARIS R.	Las Cruces, N. Mex.	55
CARPIO, GABRIEL D.	New York City	55
CLOGSTON, PERLEY W.	New York City	70
COOPER, CHARLES S.	Brooklyn, N. Y.	56
CORSON, HAROLD H.	Birmingham, Mich.	68
COTT, WILLIAM B.	Akron, Ohio	56
DORITY, ALEX O.	Cincinnati, Ohio	70
DUVALL, ARNDT J.	St. Paul, Minn.	70
EAGAR, ROBERT J.	New York City	56
EDWARDS, ROBERT F.	Chatham, N. J.	56
ELA, MOHAMED M. A.	New York City	57
GAINES, ALFREW W.	Council Bluffs, Iowa.	57
GARDNER, KENNETH E.	Baltimore, Md.	57
GARDNER, LOYDE H.	Rocky River, Ohio.	57
GOLDSMITH, MORRIS K.	Los Angeles, Cal.	58
GOPALAKRISHNA, SESHIER	Bhadravati, India.	58
GRIFFIN, ROBERT H.	San Francisco, Cal.	59
HALL, BERT A.	Nyssa, Ore.	59
HAW, ELMER P.	Ancon, Canal Zone.	68
HILTON, JOSEPH K.	Clarion, Iowa.	68
HORWATH, THEODORE J.	Newark, N. J.	59
JONES, JIM D.	Knoxville, Tenn.	59
LAMB, GILES H., JR.	Santa Maria, Cal.	70
LOFFLAND, MURREL E.	Paris, Ill.	59
MAHON, ROSS L.	Philadelphia, Pa.	59
MESSER, MERTON W.	Baton Rouge, La.	71
MILLER, LEO M.	Los Angeles, Cal.	60
MOLONY, WALTER E.	Boonton, N. J.	60
MULRYAN, DAVID E.	Rochester, N. Y.	60
NICHOLS, GEORGE H.	Atlanta, Ga.	60
NOBLE, WILLIAM H.	Parkville, Mo.	61
ORCUTT, FORREST G.	Denver, Colo.	61
OSBORNE, EDWARD G.	Seattle, Wash.	61
PANDYA, ANANT H.	Cambridge, Mass.	61
PARK, KENNETH F.	Sacramento, Cal.	62
PARKER, THOMAS C.	Springdale, Utah.	62
PEARSON, THEO. P.	New York City	62
PEDIGO, JAMES A.	Los Angeles, Cal.	62
RAEDER, ELMER W.	Boulder, Colo.	69
REIGELUTH, RAY J.	New Haven, Conn.	63
RIDDELL, JOHN C.	Little Rock, Ark.	71
ROYER, ROBERT S.	Richmond, Va.	69
RUSKIN, SAMUEL J.	Detroit, Mich.	63
SCHOEN, HARRY	Forest Hills, N. Y.	71
SETHMANN, MARTIN W.	Brooklyn, N. Y.	63
SETTLE, JOSEPH E.	Charleston, W. Va.	63
SHANE, HENRY K.	Kansas City, Mo.	64
SLOANE, HAROLD A.	New York City	64
SPRINGER, REID C.	Woodside, N. Y.	65
STOKES, CHARLES E.	Fresno, Cal.	65
STRIEBECK, WM. H., JR.	George West, Tex.	65
TAYLOR, DAVID Y.	West Medford, Mass.	66
TOMEK, ARTHUR O.	Two Rivers, Wis.	66
WELTNER, WILLIAM	Detroit, Mich.	66
WHEATON, HERBERT H.	Berkeley, Cal.	66
WILSON, ROBIN S.	Los Angeles, Cal.	67
WONG, QUEN Y.	Seattle, Wash.	67

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

The number in the center above each record indicates the serial number of the applicant for the current year, and that at the left the district in which he resides.

The abbreviations in Italics represent respectively, *TT*, Total Time; *SP*, Sub-Professional Work; *P*, Professional Work; *RC*, Responsible Charge; *D*, Design. The figure for Total Time is determined by adding one-half the time spent in Sub-Professional Work to the time spent in Professional Work. The figures showing the amount of time spent in Responsible Charge and on Design are the estimate of the Applicant. The allowance of four years for graduation or of one-half of a year for each academic year successfully completed in an engineering college without graduation is included in Total Time and Professional Work.

FOR ADMISSION

152

(3) **ABELL, DeWITT STERLING,** 17 Church St., Beacon, N. Y. (Age 35. Born Albion, Mich.) 1920 B. S. in C. E., Univ. of Mich. 1925 M. S. in S. E., Penn. State Coll. *TT 4: P 4.*—Sept. 1921 to June 1923 with Denison Univ., as Instructor (1 year) and Acting Head, Dept. of Civ. Eng. (1 year). *TT 1: P 1.*—Sept. 1923 to June 1925 Graduate Student and Graduate Asst., Dept. of Civ. Eng., Pennsylvania State Coll.—June 1925 to Jan. 1926 Tech. Asst. in San. Eng., U. S. Public Health Service, Stream Pollution Investigation Station, Cincinnati, Ohio. *TT 0.6: P 0.6.*—Dec. 1927 to July 1931 Asst. Engr., Div. of San. Eng., Ohio Dept. of Health, Columbus, Ohio, about 2 years on design, made investigations of sewerage, sewage-treatment, water-supply and water-treatment projects at schools, institutions and small municipalitiis, co-operated with officials and architects or consulting engineers on designs, checked plans, wrote reports, etc. *TT 3:6; P 3.6: RC 1: D 1.*—*TT 10.2: P 10.2: RC 1: D 1.* Refers to E. G. Bradbury, G. Champe, T. S. Johnson, F. D. Stewart, R. C. Sweeney, E. D. Walker, F. H. Waring.

153

(12) **ALLISON, WILLIAM NOEL,** 5264 Nineteenth Ave. N. E., Seattle, Wash. (Age 23. Born Golden, Colo.) 1931 B. S. in C. E., Univ. of Wash. *TT 4: P 4.*—July 1927 to Oct. 1928 Computer and Draftsman, Wallace Bridge & Structural Steel Co., detailing steel beams, bracing, etc. *TT 0.6: SP 0.6.*—Summers 1929 and 1930 successively Chainman, Washington State Highway Dept., and Clerk, Eng. Office, Dept. of Lighting, Seattle, Wash.—*TT 4.6: SP 0.6: P 4.* Refers to I. L. Collier, G. E. Hawthorn, C. E. Magnusson, C. C. More, R. G. Tyler.

154

(1) **BALABAN, EDWARD,** 1457 Broadway, New York City. (Age 38. Born New York City.) Licensed Professional Engr., New York State. 1915 B. S., and 1916 C. E., N. Y. Univ. *TT 4: P 4.*—Summer 1914 Rodman, Delaware, Lackawanna & Western R. R.—1915 to 1917 Jun. Asst. Engr., Public Service Comm., New York City, on line and grade for construction of subway section, Varick St. *TT 1.7: SP 0.2: P 1.5: RC 1: D 0.5.*—1917 to 1920 Jun. Engr. and Chf. Computer, Interstate Commerce Comm., Washington, D. C., in charge of cross sections, measurements and computations in connection with valuation of Atlantic Coast Line, Baltimore & Ohio R. R., Seaboard Air Line, etc. *TT 2.7: SP 0.2: P 2.5: RC 2.5.*—1920 to date member of firm and Pres., Balaban-Gordon Co., Inc., Contrs., in complete responsible charge and some design, on erection of Transformer Bldgs. for Dept. of Water Supply, garage for Dept. of Highways, Bronx, theatre in

Brooklyn, hospital group in Staten Island for Dept. of Hospitals, Holland Tunnel Office Bldg. and numerous others. *TT 12: P 12: RC 9.5: D 3.*—*TT 20.5: SP 0.5: P 20: RC 13: D 3.5.* Refers to B. B. Anderson, R. Bolaffio, A. Geist, J. F. Krakauer, J. N. Pease, E. E. Seelye.

155

(1) BARRABEE, ARTHUR SAMUEL, 89 Seaman Ave., New York City. (Age 23. Born Hartford, Conn.) 1932 B.S. in C. E., Univ. of Ala. *TT 4:P 4*—May 1929 to Sept. 1930 with Gibbs & Hill, Cons. Engrs., as Draftsman, Designer and Inspector on catenary electrification for Pennsylvania R. R., inspection on field construction. *TT 1: SP 0.3: P 0.7: RC 0.3: D 1.*—*TT 5: SP 0.3: P 4.7: RC 0.3: D 1.*—Refers to J. G. Allen, G. J. Davis, Jr., D. C. A. du Plantier, S. C. Houser.

156

(1) BARTLEY, ROBERT JOHN, 128 Lawrance Lane, Bay Shore, N. Y. (Age 46. Born New York City.) 1906 to 1911 Asst., and 1911 to 1916 Associate, with Eugene R. Smith, Islip, N. Y., on work in Islip, Bay Shore, Brightwaters, Oakdale, Fire Island, Bayport, etc., including concrete sidewalks (about 5 miles), curb and gutter (about 3 miles), land reclamation (215 acres), storm sewers (abouh 2 miles), two docks, a wooden pier into Atlantic Ocean, bulkheads, jetties, drainage structures, three sewage-disposal (6 000 to 12 000 gal.) tanks, breakwaters, dirt race track (½ mile), roads, channel dredging, right-of-way survey for Long Island Motor Parkway through Islip, etc.; designed wooden railway trestle (3 000 ft.), and two reinforced concrete bridges (20-ft. span, 30-ft. roadway) for an estate and superintended construction of Masonic Temple ($25 000) (given in detail in application). *TT 7.4: SP 2.7: P 4.7: D 4.7.*—1916 to date (except May 1918 to April 1919 with U. S. Army) in private practice at Bay Shore establishing official grades, designing and constructing concrete and macadam roads, concrete sidewalks, curb and gutter, storm sewers, bridges, culverts, timber bulkheading, docks, land reclamation, grading, runways and hangar design for airport, Islip, designed grade-cossing elimination, athletic fields, designed and constructed garbage-disposal incinerator, fish hatcheries with canals, weirs and ponds (6 acres), a ferry terminal, harbor construction, mosquito extermination work involving construction 50 miles of ditch, maintenance, etc. (given in detail in application); Cons. Engr., Islip Town Planning and Zoning Comm., Engr. Member, Board of Health, Suffolk County. *TT 15.1: P 15.1: RC 15.1: D 15.1.*—*TT 22.5: SP 2.7: P 19.8: RC 15.1: D 19.8.* Refers to F. J. Biele, H. B. Cleveland, C. F. Morse, A. J. Provost, Jr., A. O. Smith, E. R. Smith, A. V. D. Wallace, Jr., I. Walzer.

157

(2) BEHLEY, RUDOLPH JOHN, 31 Prospect St., Milford, Conn. (Age 30. Born East Norwalk, Conn.) Sept. 1919 to June 1921 Student in surveying, etc., Wesleyan Univ.—July 1921 to Aug. 1924 Rodman, Transitman and Party Chf., Dade County Surveying Co., Miami, Fla., on subdevelopment. *TT 2.2: SP 1: P 1.2: RC 1*—Sept. 1924 to March 1927 Timekeeper, Foreman and Asst. Supt., P. J. Davis Constr. Co., Miami, laid out buildings, walls and footings, gave grades, explained plans, etc. *TT 1.8: SP 0.8: P 1: RC 0.5.*—April 1927 to Oct. 1928 Foreman, Estimator and Engr., J. A. Corbet Co., Syracuse, N. Y., estimating and superintending building of roads, railroads, bridges and sewers, etc. *TT 1.5: P 1.5: RC 0.8.*—Nov. 1928 to June 1929 Engr., Transkutan Sanitoriums, Inc., Bernardsville, N. J., remodeling private estate for hospital purposes, including reconstructing buildings, designing and constructing small dam and disposal plant, water lines, sewers, etc. *TT 0.5: P 0.5: RC 0.5.*—July 1929 to date Constr. Engr., Sperry & Treat Co., New Haven, Conn., being Res. Engr., work includes layout of building and grades, explanation of plans, etc. *TT 2.6: P 2.6: RC 2.6.*—*TT 8.6: SP 1.8: P 6.8: RC 5.4.*—Refers to J. S. Allen, J. Andrews, III, J. J. Hurley, C. J. Messier, H. G. Wynne.

158

(1) BERSIG, WALTER CHARLES, 1343 East Twenty-Ninth St., Brooklyn, N. Y. (Age 22. Born Brooklyn, N. Y.) 1931 B. S. in C. E., Cooper Union. *TT 4: P 4.*—June 1925 to Feb. 1931 (while student) with New York Edison Co., New York City, as Clerk, Tracer of mechanical and structural drawings, and Draftsman on mechanical layouts, small structural steel, and (after Jan. 1930) with Empire City Subway Co., Ltd., New York City, as Rodman, estimating, drafting, etc., and Asst. Engr. on construction of telephone subway conduits, in charge of laying out and supervising work, etc.—Feb. 1931 to date (until June 1931 while student) Eng., Asst., Board of Transportation, New York City, surveying, computing and drafting, also inspecting, new rapid transit subway system. *TT 0.4: SP 0.4*—*TT 4.4: SP 0.4: P 4.* Refers to J. Farhi, F. E. Foss, H. Holbrook.

159

(1) **BILLING, JOHN MORTON,** Public Works Dept., Singapore, Straits Settlements. (Age 29. Born Blackpool, England.) 1922 B. Sc. (Eng.), Univ. of London. *TT 4: P 4.*—Jan.-May 1924 Lecturer, Southend Evening Technical School. *TT 0.4: P 0.4: RC 0.4.*—June 1924 to Feb. 1926 Jun. Asst., County Borough Engr.'s Office, Corporation of Southend-on-Sea, England, in Bldgs. Inspectors' and Town Planning Secs. *TT 1.3: SP 0.3: P 1: RC 1.*—Feb. to Sept. 1926 Asst., Borough Engr's Office, Crewe, England, on surveys, etc. *TT 0.7: P 0.7: RC 0.7.*—Sept. 1926 to date Asst. Engr., Public Works Dept., Government of Malay States (Federated), having charge (under Senior Executive Engr.) of dwellings, public buildings, sanitation, sewerage, drainage, water-works, mechanical and steam, streets, wharves, sea-walls, reclamation and maintenance (total 1927-1930 $1 964 000). *TT 5.4: P 5.4: RC 4.9: D 2.9.*—*TT 11.8: SP 0.3: P 11.5: RC 7: D 2.9.* Refers to A. C. Alvarez, C. G. Hyde. (Applies in accordance with Sec. 1, Art. I, of the By-Laws).

160

(1) **BRODKIN, NATHAN DANIEL,** 3400 Wayne Ave, New York City. (Age 28. Born New York City.) Licensed Prof. Engr., New York State. 1924 B. S. in Civ. Eng., Cooper Union (Night School). *TT 4: P 4.*—June 1919 to March 1920 Draftsman with M. Schwartz, Archt., New York City, on plans and designs for buildings and alterations to buildings.—March to Sept. 1920 Estimator, Atlantic Terra Cotta Co., New York City, on quantity and cost estimates.—Nov. 1920 to March 1921 Chemical Laboratory Asst., Rockefeller Inst. for Medical Research, chemical research work in Organic Chemistry.—March 1921 to Feb. 1922 Draftsman, Dept. of Plant and Structures, Designs Div., New York City, tracing, drafting and elementary design for bridges, ferry terminals, public markets, incinerators and garages.—Feb. 1922 to Jan. 1924 Rodman, Dept. of Parks, New York City, acting as Rodman, Leveler and Transitman, and as Draftsman or Computor inspecting construction of reinforced concrete bulkhead along Harlem River Driveway.—Jan. 1924 to date with Board of Transportation, New York City, as structural Draftsman, Asst. Squad Leader, and (after Aug. 1926) Structural Designer, Grade 4; until June 1925 on structural and reinforced concrete drafting and design, and since June 1925 determining alignments and grades and in responsible charge of preparation of drawings and designs, including projects in Brooklyn and Manhattan (some named in application) for subways and elevated structures, checks designs, decking and construction, advises engineers in field, etc.; at present on third alternate design for proposed Sixth Ave. Subway, Christopher to 18th St., designed supports of elevated columns, overhead steam railroads, street surface railroads, existing subways, etc., reinforced concrete arch under private property (designed to support building loads), reconstruction of Hudson & Manhattan R. R. in Sixth Ave. (during operation) and track flexing at 8th Ave., Manhattan, and at Steinman St. and Northern Boulevard, Queens. *TT 7.6: P 7.6: RC 6.8: D 6.8. TT 11.6: P 11.6: RC 6.8: D 6.8.* Refers to F. B. Barshell, R. Kosches, J. H. Quimby, A. I. Raisman, F. Viola, T. F. Weiss.

161

(15) **BURN, PARIS RUSSELL,** Las Cruces, N. Mex. (Age 34. Born Beaufort, S. C.) 1918 C. E., Univ. of S. C. *TT 4: P 4.*—March 1918 to July 1919 2d Lieut., Engrs., U. S. Army, commanding Co. C, 546th Engrs., A. E. F., on highway and railroad construction. *TT 1.2: P 1.2: RC 1.2.*—July 1919 to Sept. 1920 Supt. of Constr. and Post Engr., with Lt.-Col. F. G. Chamberlain at Ft. Huachuca, Ariz., in charge of construction and design of buildings, water-works, sanitary sewers, roads, bridge and dam (total about $700 000). *TT 1.3: P 1.3: RC 1.3: D 1.3.*—Sept. to Nov. 1920 Supt. of Constr. and Engr., with F. M. Hayner, Las Cruces, N. Mex., in charge of construction of steel hangars, buildings, water-works, sanitary sewers and lighting system for airdromes, Marfa, Tex. *TT 0.2: P 0.2: RC 0.2.*—Nov. 1920 to date member of firm and Gen. Mgr., Hayner & Burn, Engrs. and Contrs., on construction of engineering projects in towns and cities of New Mexico, Texas, Arizona and Colorado, installing water-works, sanitary and storm sewer systems, gas lines, pavements, roads, bridges, sewage-treatment, disposal and water-purification plants, etc. (total several million dollars); also worked on design and redesign of systems and structures. *TT 11.2: P 11.2: RC 11.2: D 11.2.*—*TT 18: P 18: RC 14: D 12.5.* Refers to E. L. Barrows, G. F. Conroy, H. B. Elmendorf, P. S. Fox, V. J. Von Schoeler, W. H. W. Yeo.

162

(1) **CARPIO, GABRIEL DUMLAO,** 257 West Fourteenth St., New York City. (Age 26. Born Paoay, Ilocos Norte, Phillippine Islands.) 1931 B.S. in Arch. Eng., Univ. of Ill. *TT 4: P 4.*—Refers to N. D. Morgan, C. E. Palmer.

163

(1) COOPER, CHARLES SAMUEL, 183 Ocean Parkway, Brooklyn, N. Y. (Age 30. Born Minsk, Russia.) 1924 B. C. E., Northeastern Univ.—June 1920 to June 1924 (Co-operative work) until June 1921 as Transitman with W. C. Bates, Jamaica Plain, Mass., drafting, computing and field work on surveys and building layouts, and after June 1921 Asst. Engr. with C. H. Gannett, Boston, Mass., being Chf. of Party on surveys and building construction, in charge of a real estate development, supervising drafting, computing and field work.—June 1924 to June 1925 Asst. Engr., Shumco Eng. Co., Boston, Mass., in charge of remodeling Central Theatre, Revere, Mass. *TT 1.: P 1.*—Aug. to Nov. 1925 Inspector, New York State Bridge & Tunnel Comm., in charge of eight-hour shift on Hudson River dredging for Holland Vehicular Tunnel. *TT 0.2: P 0.2.*—Nov. 1925 to Feb. 1927 Jun. Engr., Trenor Land Co., New Rochelle, N. Y., in charge of Drafting Room, supervising preparation of topographical maps, computations and plans for street layouts, sewers and concrete highways for development of Wykagyl Park. *TT 1.3: P 1.3.*—Oct. 1927 to Jan 1928 Asst. Engr. with M. J. Fielder, New York City, in charge of Drafting Room, supervised preparation of composit map of Spuyten Duyvil Creek. *TT 0.3: P 0.3.*—Oct. 1928 to April 1929 Topographical Draftsman, Board of Transportation, New York City, computed data for column foundations, steel erection and subway alignment. *TT 0.5: P 0.5.*—May to Oct. 1928 Asst. Engr., Div. of Survey and Design, and April 1929 to date Topographical Draftsman, Bureau of Highways, Brooklyn, N. Y., preparing plans and specifications for paving and repaving streets in Brooklyn and for highway improvements, since May 1930 being Asst. in charge of Drafting Room, Div. of Survey and Design, having complete charge in absence of superior. *TT 3.3: P 3.3: RC 1.*—*TT 6.6: P 6.6: RC 1.* Refers to H. B. Alvord, G. Berry, T. B. Brogan, C. S. Ell, J. C. O'Dea, W. R. Tenney.

164

(9) COTT, WILLIAM BELLMAN, 182 Westwood Ave., Akron, Ohio. (Age 23. Born Pittsburgh, Pa.) 1932 B. S. in Civ. Eng., Univ. of Ill. *TT 4: P 4.* Refers to W. C. Huntington, G. W. Pickels.

165

(1) EAGAR, ROBERT JOSEPH, 424 West One Hundred Fifty-fourth St., New York City. (Age 42. Born New York City.) 1929 LL.B., N. Y. Univ. 3 years student, Extension Course, Columbia Univ., concrete and steel design and drafting—1910 to 1913 with Frederick E. Glasser, Civ. Engr. and Surveyor, first as Rodman and finally Chf. of party, on surveys, design and construction of sewers and highways. *TT 1.5: SP 1.5.*—1913 to 1917 Jun. Asst. Engr., Public Service Comm., being Transitman, Chf. of Party and finally in charge of about ½ mile of subway construction (under Sec. Engr.). *TT 4: P 4: RC 1.5.*—April 1917 to Aug. 1919 with Constr. Q. M. Corps, U. S. Army, first at Training Camp, Plattsburg, N. Y., Aug. 1917 to Aug. 1918 Second Lieut. and after Aug. 1918 First Lieut., on construction of Camp Upton, and in charge of reconditioning country roads (7 miles) at Camp McClellan, Ala., involving designing and building small timber bridges and culverts, also road grading; later with A. E. F., France. *TT 1: P 1: RC 1.*—1920 to date in private practice as Engr. and Surveyor, surveying, estimating, computing quantities, checking and making reports, giving court testimony, etc., and occasionally designing retaining and sea walls, and small sections of sewers. *TT 11: P 11: RC 11.*—*TT 17.5: SP 1.5: P 16: RC 13.5.* Refers to A. E. Clark, J. J. Cope, A. J. Daino, C. F. Giraud, G. Paaswell, P. Sander, A. V. Sheridan.

166

(1) EDWARDS, ROBERT FIELDING, 82 Watchung Ave., Chatham, N. J. (Age 35. Born Brooklyn, N. Y.) 1917 C. E., Cornell Univ. *TT 4: P 4.*—Aug. to Sept. 1917 Draftsman in machine shop, Ford Instrument Co., New York City. *TT 0.1: SP 0.1.*—Oct. 1917 to May 1920 with Water Resources Branch, U. S. Geological Survey, 6 months as Asst. Classifier surveying and mapping lands in Colorado and New Mexico, then Asst. Engr. on stream gauging in Northern Montana, field and office work. *TT 2.4: SP 0.2: P 2.2: RC 2.2.*—June to Dec. 1920 with County Surveyor's Office, Fergus County, Mont., surveying and mapping. *TT 0.3: SP 0.3.*—Jan. 1921 to Jan. 1923 Asst. Engr., Federal Power Comm., Washington, D. C., had charge of land reserves and made field surveys and reports on power possibilities of numerous rivers in northwestern states. *TT 2: P 2: RC 2: D 2.*—Feb. 1923 to June 1924 Computer and Designer, Southern Power Co. (Duke Power Co.), Charlotte, N. C., made hydraulic studies, designed and checked hydro-electric and steam-electric power plant structures. *TT 1: SP 0.3: P 0.7: RC 0.7: D 1.*—Sept. 1924 to March 1929 with Public Service Production Co., Newark, N. J. (now United

Engrs. & Constrs., Inc., Philadelphia, Pa.) until Jan. 1929 as Designer and Checker, made original plant layouts, designed and checked structural and mechanical parts of steam-electric power, cement and gas plants, indoor and outdoor substations and commercial offices, and after Feb. 1929 as Designer, made layouts and preliminary designs for new Pennsylvania R. R. Station in West Philadelphia. *TT 4: SP 0.4: P 3.6: RC 3.5: D 4.*—March 1929 to date Asst. Engr., Transit Bureau, Newark, designing, checking and supervising design and construction details of city railway and appurtenant structures and Haynes Ave. Bridge, Newark. *TT 3: P 3: RC 3: D 3*—*TT 17: SP 1.5: P 15.5: RC 11.4: D 7.1.* Refers to C. L. Crandall, R. W. Davenport, B. E. Jones, R. Smillie, R. von Fabrice, E. Welle, L. S. Whippen.

167

(1) ELA, MOHAMED MAHMOND ABOUL, 170 Broadway, New York City. (Age 27. Born Alexandria, Egypt.) 1927 B. Sc. Tech., Mun. and San. Eng., Manchester (England) Univ. *TT 4: P 4.*—Nov. 1927 to Feb. 1928 Asst. Engr., Water-Works Dept., Mun. Sec., Ministry of Interior, Cairo, on design of pipe lines and water towers. *TT 0.3: P 0.3: D 0.3.*—March 1928 to date Asst. Engr., Sewerage Dept., Alexandria Municipality, Egypt, on sanitary inspection and approval of building permits, technical assistance in court cases, designing sewers for four districts (8 000 to 12 000 ft. long), and Hadara trunk sewer (about $73,000); in responsible charge of construction of Gheit El Enab open drain with retaining wall on one side, outfall extending 120 yd. into lake (about $55 000), also of Sidi Gaber pumping station in sandy soil 14 ft. below ground water, requiring sheet piling protection for railway embankment; 4 months in charge of construction work, and in supervisory charge of main sewage-pumping station of city; since June 1931 studying sewerage and sewage treatment practice in various countries, since Aug. 1931 being with Fuller & McClintock, New York City, assisting on reports to Hackensack Valley Sewerage Comm. on means of eliminating pollution of Hackensack River and to City of Hagerstown, Md., on extension to local treatment works. *TT 4: P 4: RC 2.8: D 1.*—*TT 8.3: P 8.3: RC 2.8: D 1.3.*—Refers to G. E. Barnes, W. Donaldson, G. W. Fuller, D. W. Howes, E. W. Whitlock.

168

(16) GAINES, ALFREW WALLACE, 320 North Second St., Council Bluffs, Iowa. (Age 50. Born Tecumseh, Nebr.) Sept. 1900 to June 1902 Eng. Student, Univ. of Nebraska. *TT 1: P 1.*—Feb. 1904 to Jan. 1905 and June 1923 to March 1926 in private practice with C. L. Huff, in Iowa, drainage, highway and land surveys, designs, construction of sewers, bridges and culverts. *TT 3.8: P 3.8: RC 2.8: D 2.8.*—Feb. 1916 to Jan. 1920 County Engr., Harrison County, in charge of highway construction. *TT 4: P 4: RC 4: D 4.*—March 1926 to May 1928 Res. Engr., Iowa State Highway Comm., in charge of construction and of parties on surveys and plans for highway work (approx. 50 miles). *TT 2.2: P 2.2: RC 2.2: D 2.2.*—May 1928 to date Constr. Engr., Geo. W. Condon Co., Omaha, Nebr., checking designs, estimating bidding and having general supervision of contracts for railway, highway and drainage construction ($2 000 000 to $5 000 000 annually). *TT 3.8: P 3.8: RC 3.8: D 3.8.*—*TT 14.8: P 14.8: RC 12.8: D 12.8.* Refers to C. C. Coykendall, O. W. Crowley, R O. Green, C. L. Huff, F. H. Mann, L. M. Martin, J. H. Mayne.

169

(5) GARDNER, KENNETH ELMER, 407 Pontiac Ave., Brooklyn, Baltimore, Md. (Age 24. Born Brooklyn, Baltimore, Md.) 1930 B. E., Johns Hopkins Univ. *TT 4: P 4.*—June 1930 to Feb. 1931 Engr.-Draftsman, Buffalo, Rochester & Pittsburgh R. R. Co., construction accounting, drafting, assisting on estimates and figuring bridge stresses. *TT 0.4: SP 0.4.*—April 1931 to Jan. 1932 Engr., Forbes-Murphy Constr. Co., Baltimore, Md., general layout in bridge and gasholder work, keeping time and cost records and estimating quantities of material. *TT 0.4: SP 0.4.*—*TT 4.8: SP 0.8: P 4.* Refers to T. F. Comber, Jr., T. F. Hubbard, F. W. Medaugh, W. F. Pond, J. T. Thompson.

170

(9) GARDNER, LOYDE HARRISON, 20545 Morewood Parkway, Rocky River, Ohio. (Age 39. Born Atlanta, Ill.) 1916 C. E., Ohio Northern Univ. *TT 4: P 4.*—1916 to 1920 (except 6 months 2d Lieut., U. S. Field Artillery) with Carnegie Steel Co., Youngstown, Ohio, over 1½ years as Draftsman on maps, establishing railroad and street grades, platting river cross-sections, taking river soundings, designing, estimating and laying out relocation of railroad tracks and trestles for an extension, water lines, sanitary and storm sewers, pavements, reinforced concrete fences, sanitary and welfare buildings, including drinking fountains, locker and shower rooms, etc., for proposed coke ovens and by-product plant, etc., 1⅔ years Asst. to Field Engr., at McDonald (Ohio) Mills being in charge of rein-

forced concrete work, supervising and laying out reinforcing, assembling material, making sketches for and supervising rod bending and placing, designing footers for building columns on fill, covering mill-building foundations, coal-pulverising plant, hoppers, bins, concrete sewers, water-pipe and electric conduits, subway (2 000 ft.), transformer station, supervising construction of pump house and reinforced concrete intake at river, and 9 months Special Engr., Operating Dept., McDonald Mills, making coal pulverizor and dryer efficiency tests, collecting data, determining sizes and weights of small hoops, bars and rods, rolled on small mills, etc. *TT 4.1: SP 0.3: P 3.8: RC 2.4: D 1.*—1920 to 1921 Designer, Lackawanna Bridge Co., Buffalo, N. Y., designing and detailing stairs on skip bridge for blast furnace, roof trusses, tie rods, steel ladders and walks for furnace building, bents, crane rail girders, upper chord lateral bracing of chamber building of chemical plant, foundation plan of crane runway for power plant, rail borings and splice plates for rail connections, etc. *TT 0. 4: P 0.4: D 0.4.*—1921 to 1925 Asst. Constr. Engr., Carnegie Land Co., Pittsburgh, Pa., on housing and community developments, including estimates for houses, streets, water, sewer, electric-light and gas lines, designing reinforced concrete and structural steel for filter plant (400 000 gal.), and designing, estimating and supervising construction of small reinforced concrete reservoir and addition to pump house, valuation and cost reports, etc. *TT 3.8: P 3.8: RC 0.8: D 3.*—1925 to 1927 Field Engr., Portland Cement Association, Chicago, Ill., teaching and demonstrating application of water-ratio method of proportioning concrete, assisting county, city, village and consulting engineers in road, street and other improvement problems involving use of concrete, inspecting construction of pavements, buildings, etc. *TT 1: SP 1.*—1927 to date Dist. Engr., Ohio Corrugated Culvert Co., Middletown, Ohio, in charge of branch office at Cleveland, promoting and selling, and assisting state, county, city and village engineers in designing, estimating cost, and supervising installation of Armco drainage products, such as paved invert pipe for sewers, culverts, stream enclosures, also spiral welded smooth pipe for pressure lines, and metal cubbing for retaining walls. *TT 4: P 4: RC 4.*—*TT 16.9: SP 1.3: P 15.6: RC 7.2: D 4.4.* Refers to A. A. Burger, A. Diefendorf, F. L. Gorman, R. F. MacDowell, G. F. Pfeiffer, C. L. Wooldridge.

171

(11) GOLDSMITH, MORRIS KING, 3714 Mullen Way, Los Angeles, Cal. (Age 28. Born New York City.) 1924 B. S. in Civ. Eng., Cal. Inst. Tech. *TT 4. P 4.*—June to Dec. 1924 Material Clerk, P. J. Walker Co., in charge of expediting material on building construction. *TT 0.2: SP 0.2.*—Jan. 1927 to Jan. 1928 Mgr., Temescal Groves, Inc., Corona, Cal., in charge of water development, irrigation, land reclamation. *TT 1: P 1: RC 1.*—Dec. 1924 to Jan. 1927 and Jan. 1928 to Jan. 1932 with H. M. Baruch Corporation, Los Angeles, Cal., until Jan. 1926 as Estimator, Draftsman and Timekeeper on estimates, quantity surveys and drafting for millwork, then (1 year) Field Engr., and after Jan. 1928 Engr., on surveys, estimates, inspection, cost analyses, plant, form, structural and foundation procedure designs, co-ordination of crafts, and acting as Asst. Supt. on construction. *TT 5.5: SP 0.5: P 5: RC 1: D 1.*—*TT 10.7: SP 0.7: P 10: RC 2: D 1.* Refers to S. B. Barnes, R. V. Labarre, R. R. Martel, F. Thomas, H. C. Whittlesey.

172

(1) GOPALAKRISHNA, SESHIER, Mysore Iron Works, Bhadravati, Mysore State, India. (Age 44. Born Dharapuram, Coimbatore Dist., South India.) 1907 received certificates of Sub-Overseer, Surveyor and Draftsman, Eng. School, Mysore Govt. Studied Architectural Eng., International Correspondence School, Ltd., London.—Dec. 1913 to July 1916 Div. Draftsman, Public Works Dept., Nagapore Div. No. 2, Central Provinces, in charge of Drawing Office. *TT 1.3: SP 1.3.*—Nov. 1916 to Sept. 1918 Drainage Scheme Draftsman, Mysore Govt. Public Works Dept., Bangalore, designed Bangalore public market, drainage scheme equipment and layout of sanitary extensions to city. *TT 0.9: SP 0.9.*—Sept. 1918 to July 1919 Chf. Draftsman, Mysore Iron Works Scheme, made investigations, designed temporary staff quarters and permanent residential buildings, layout of town and location of site for plant, etc., at Bhadravati. *TT 0.9: P 0.9: D 0.9.*—July 1919 to date Chf. Draftsman, Mysore Iron Works, Bhadravati (state owned), comprising saw-mill, wood-carbonizing, blast-furnace and ore-crushing plants, machine and pattern shops, foundry, pipe foundry with power plant, boiler and pump houses, and chemical plant for refining byproducts of wood distillation; designed layout of new town, its drainage scheme, roads, culverts, bungalows for officers, staff quarters, numerous details for plant construction, offices, laboratory, stores, track layout, etc. (cost of plant, including 60 miles of tramway, about $10 000 000); also designed tar-distillation plant, and details of cellar arrangements, foundations and structural steel work for centrifugal system, getting out details for fitting up equipments for Alcohol Refinery Plant. *TT 12.5: P 12.5:*

RC 12.5: D 12.5.—*TT 15.6: SP 2.2: P 13.4: RC 12.5: D 13.4.*—refers to L. M. Allison, F. W. Claflin, C. R. Ege, N. K. Fougner, C. H. Fuller, G. L. Gaiser, E. A. Gibbs, E. V. Owen.

173

(13) GRIFFIN, ROBERT HARDY, Ferry Bldg., San Francisco, Cal. (Age 22. Born Charleston, Ill.) 1931 B.S. in Civ. Eng., Cal. Inst, Tech. *TT 4: P 4.*—June 1931 to date Jun. Engr., U. S. Forest Service, during summer being Chf. of topographic mapping party and winter drafting, etc., on bridge work, etc. *TT 0.3: SP 0.3.*—*TT 4.3: SP 0.3: P 4.* Refers to J. H. Lawrence, R. R. Martel, W. W. Michael, F. Thomas.

174

(12) HALL, BERT ANDREW, Nyssa, Ore. (Age 38. Born Montrose, Colo.) Licensed Civ. Engr., State of Idaho.—Dec. 1915 to June 1917 student, Valparaiso Univ.—Sept. 1917 to March 1919 with U. S. Army, until Dec. 1918 with Infantry, then Master Engr., Senior Grade, 213th Engrs. *TT 0.2: P 0.2.*—Aug. 1912 to Dec. 1915, June to Sept. 1917 and March 1919 to date with U. S. Bureau of Reclamation, until Sept. 1917 as Rodman, Levelman and Instrumentman, on location and construction of irrigation and drainage canals and on water measurements and records; March 1919 to March 1920 in charge of field party, on topographic surveys, subdivision and retracements; March 1920 to Aug. 1929 Asst. Engr., on office engineering, right-of-way computations, design of water and sewer systems, sidewalk and street grades for new townsite of American Falls, Idaho, preliminary investigations of Island Park Reservoir, Idaho, location of 30 miles of 33 000-volt transmission line, and in charge of inspection and field engineering on American Falls Dam, Idaho, and Owyhee Dam, Ore., and six miles of construction railroad; since Aug. 1929 Associate Engr., in charge of all inspection and field engineering on Owyhee Dam and appurtenant works, upper 1 8/10 miles of irrigation tunnel No. 1. *TT 14.4: SP 1.7: P 12.7: RC 7.8.*—*TT 14.6: SP 1.7: P 12.9: RC 7.8.* Refers to F. A. Banks, B. Dibble, D. C. Henny, J. L. Savage, B. W. Steele.

175

(1) HORWATH, THEODORE JOSEPH, 920 Bergen St., Newark, N. J. (Age 24. Born Newark, N. J.) 1931 B. S. C. E., Purdue Univ. *TT 4: P 4.*—June 1927 to Feb. 1928 Draftsman, New Jersey Bell Telephone Co., Newark, N. J., on street and conduit layout, tracing and drafting. *TT 0.3: SP 0.3.*—June 1928 to Feb. 1929 Jun. Engr., New Jersey State Highway Comm., Newark, acting as Instrumentman. *TT 0.4: SP 0.4.*—*TT 4.7: SP 0.7: P 4.* Refers to W. J. Henderson, S. C. Hollister, G. P. Springer.

176

(10) JONES, JIM D, 823 South Fifteenth St., Knoxville, Tenn. (Age 25. Born Troy, Tenn.) 1931 B. S. in C. E., Univ. of Tenn. *TT 4: P 4.*—Aug. to Dec. 1927, March to June 1928, Sept. to Dec. 1928, March to June 1929, Aug. to Dec. 1929, March to June 1930 and Sept. to Dec. 1930 (co-operative work) Rodman, Tennessee State Highway Dept., on triangulation, staking out overflow bridges, on location party, set slope stakes, calculated overhaul and yardage for earth work, on final estimate of concrete road, staked out culverts and bridges and on final estimates of concrete highway and bridges.—*TT 4: P 4.* Refers to N. W. Dougherty, H. H. Hale.

177

(8) LOFFLAND, MURREL EDWARD, 415 East Court St., Paris, Ill. (Age 23. Born Worthington, Ind.) 1930 B. S., Rose Pol. Inst. *TT 4: P 4.*—Sept. 1930 to date Jun. Engr., State Dept. of Public Works and Bldgs., Div. of Highways, Illinois, acting as Draftsman and Inspector. *TT 0.7: SP 0.7.*—*TT 4.7: SP 0.7: P 4.*—Refers to C. H. Apple, R. E. Hutchins, R. L. McCormick.

178

(4) MAHON, ROSS L'ESTRANCE, Paschall Station Post Office, Philadelphia, Pa. (Age 41. Born Detroit, Mich.) 1912 B. S. in Eng., Univ. of Mich. *TT 4: P 4.*—July 1912 to Feb. 1913 with Chas. Butters, Min. Engr., San Francisco, Cal., and Feb. to July 1913 with a mining company on Comstock Lode, Virginia City, Nev. *TT 0.8: SP 0.3: P 0.5.*—May 1917 to July 1919 1st Lieut. and Capt., Engr. Corps, U. S. Army, 9 months with A. E. F. at present Major, Engr. Reserve.—July 1913 to Nov. 1931 (except May 1917 to July 1919) with Pelton Water Wheel Co., San Francisco, first as Sales Engr., July 1926 to Jan. 1929 Asst. Sales Mgr., and after Jan. 1929 Vice-Pres. and Gen. Mgr., being responsible for and supervising engineering, production and sales, supervising research and testing work; work of Company included large hydraulic turbines and impulse water wheels (largest 40 000 h.p.), deep well and other centrifugal pumps, water-wheel governors, valves, machine work, etc. *TT 18.4: P 18.4: RC 12.3: D 6.1.*—At present with Baldwin-South-

wark Corporation, Philadelphia, Pa.—*TT 23.2: SP 0.3: P 22.9: RC 12.3: D 6.1.* Refers to W. Dreyer, J. D. Galloway, D. C. Henny, W. L. Huber, E. C. Hutchinson, W. E. Jessup, A. H. Markwart.

179

(11) MILLER, LEO MARC, 5611 Carlton Way, Los Angeles, Cal. (Age 29. Born Los Angeles, Cal.) 1925 B. S. in C. E., Cal Inst. Tech. *TT 4: P 4.*—July 1919 to Jan. 1921 Chainman and Rodman on surveys, 1 year with Howard McCurdy, Vernon, Cal., then with Geo. N. Adams, Los Angeles, Cal. *TT 0.7: SP 0.7.*—June 1925 to May 1926 Draftsman and Inspector, City of Vernon, on streets, sewers, water supply, etc. *TT 0.5: SP 0.5.*—May 1926 to Jan. 1927 Structural Designer and Sales Engr., Downs Bros., Los Angeles, designing belt conveyors, truck turntables, timber derricks and riggers' equipment. *TT 0.5: SP 0.2: P 0.3.*—Jan. 1927 to Oct. 1931 Civ. Engr., Draftsman and Squad Leader, Union Oil Co. of California, designing and investigating reinforced concrete, timber and steel structures, engineering studies and reports, resident supervision on construction of 80-mile pipe line and pumping plant, King County, Cal., and Res. Engr. on wharf construction, Edmonds, Wash. *TT 3.8: P 3.8: RC 2.8: D 1.*—Oct. 1931 to date Civ. Eng. Designer, Metropolitan Water Dist. of Southern California, structural and general civil engineering work. *TT 0.3: P 0.3 RC 0.3.*—*TT 9.8: SP 1.4: P 8.4: RC 3.1: D 1.* Refers to J. Hinds, F. W. Karge, R. J. Reed, O. J. Schieber, F. Thomas, W. E. Whittier.

180

(1) MOLONY, WALTER EVERETT, 437 Cornelia St., Boonton, N. J. (Age 26. Born Brooklyn, N. Y.) 1927 C. E., Rens. Pol. Inst. *TT 4: P 4.*—July 1927 to Dec. 1929 Asst. Engr., Cord Meyer Development Co., being Constr. Clerk, Lines and Grades Engr., and on quantity takeoffs on apartment and commercial building construction. *TT 2.4: P 2.4.*—Dec. 1929 to date with George A. Johnson, Cons. Engr., New York City, until Nov. 1931 as Asst. Res. Engr. giving lines and grades, detailing changes in design, computing quantities, and inspecting interceptor sewers and treatment plant ($2 000 000), Norwalk, Conn., and since Nov. 1931 Field Engr. on topographic surveying and stream-flow studies for proposed reservoir for Jersey City, N. J. *TT 2.2: P 2.2.*—*TT 8.6: P 8.6.* Refers to A. Alexander, F. S. Childs, R. M. Genthon, G. A. Johnson, T. R. Lawson.

181

(3) MULRYAN, DAVID EDWARD, 426 Bay St., Rochester, N. Y. (Age 37. Born Rochester, N. Y.) Prof. Engr., New York State.—May 1911 to May 1914 (while student) Chainman, Rodman and Tracer with J. C. Ryan, Civ. Engr. and Surveyor, Rochester, N. Y., on surveys, subdivisions, sewers, walks and grading.—May 1914 to date with City of Rochester on construction of sewers, pavements, walks, etc., until Oct. 1916 as Chainman and Draftsman, Oct. 1916 to Feb. 1925 Transitman, Feb. 1925 to Dec. 1931 Chf. of Party and since Dec. 1931 Asst. Engr.; since Oct. 1916 also made private surveys. *TT 12.4: SP 5.3: P 7.1: RC 6.* Refers to E. A. Fisher, H. L. Howe, I. E. Matthews, E. L. Oliver, C. A. Poole, J. F. Skinner, L. B. Smith.

182

(10) NICHOLS, GEORGE HENRY, 768 Cascade Ave. S. W., Atlanta, Ga. (Age 27. Born Apalachicola, Fla.) 1928 Certificate in Civ. Eng. (evening school), Ga. School Tech.—June 1922 to Nov. 1924 Chainman and Transitman for O. F. Kauffman, Civ. Engr., Atlanta, Ga., on surveys, subdivision development, topography and levels. *TT 1.2: SP 1.2.*—Nov. 1924 to April 1925 Engr., Kackley & Reynolds, Civ. Engrs., Daytona, Fla., 2000-acre topography survey and subdivision development. *TT 0.3: SP 0.3.*—April 1925 to May 1926 Bookkeeper, Biscayne Eng. Co., Miami, Fla., blueprinting, handling engineering supplies, etc.—May to Dec. 1926 Engr. for O. F. Kauffman, Civ. Engr., Atlanta, on subdivision development, including road locations, levels, profiling, computing grades and earth quantities, slope staking, lateral sewer ditching and construction, supervising and inspecting concrete roads and sidewalks, construction estimates. *TT 0.5: P 0.5: RC 0.5.*—Dec. 1926 to Oct. 1927 Draftsman with County Surveyor, Fulton County, Atlanta, Ga., surveying and mapping atlas of County. *TT 0.9: P 0.9.*—Oct. 1927 to Jan. 1931 Transitman (2 years), Engr. (6 months), Draftsman (9 months), Constr. Dept., City of Atlanta, precise traverse control, computing traverse closures, surveying, mapping property of City. *TT 3.1: P 3.1: RC 0.5.*—Jan. to Aug. 1931 Surveyman (private practice) on surveys, levels, preliminary subdivision designs, mapping and profiling. *TT 0.7: P 0.7.*—Aug. 1931 to date Draftsman, B. M. Hall & Sons, Civ. & Min. Engrs., Atlanta, Ga., topographic drafting, computing traverse closures and Transitman on topography, surveys, etc. *TT 0.4:* P 0.4.—*TT 7.1: SP 1.5: P 5.6: RC 1.* Refers to B. M. Hall, Jr., M. R. Hall, W. E. Hall, J. de B. Kops, W. E. Reynolds.

183

(16) NOBLE, WILLIAM HENRY, Parkville, Mo. (Age 55. Born State Center, Iowa.) 1909 B. S. in Civ. Eng., 1927 C. E. and 1928 M. S. in Civ. Eng., State Coll. of Wash. *TT 4: P 4.*—Summers Rodman and Chainman, Northern Pacific R. R. (1905) Deputy County Engr., Yakima County, Wash., being Chf. of Field Party, Drainage Dist., surveys and mapped work (1907), and Transitman, U. S. Reclamation Service, on location of laterals, Yakima Irrigation Project (1908).—May to Oct. 1909 Agt., Office of Experiment Stations, Dept. of Agriculture, in charge of irrigation investigation and other exhibits, Alaska-Yukon-Pacific Fair, Seattle, Wash. *TT 0.4: P 0.4: RC 0.4.*—July 1910 to Dec. 1927 in private practice as Civ. Engr., Pateros, Wash., being Deputy County Engr., on highway location and construction for Washington State and Okanogan and Douglas Counties (2 years); Chf. of Party on location of Methow Valley R. R. and on construction for Biles-Coleman Lumber Co. (10 months); Mineral Surveyor on official surveys in Washington State (9 months); Town Engr., Pateros, and (part of time) for Brewster and Bridgeport, Wash. on water supply and street improvement (2 2/5 years); Irrigation Engr., on construction, design and investigation for six local irrigation districts in Okanogan and Douglas Counties (3 years) and on miscellaneous surveys, construction and estimates (2 7/10 years). *TT 11.6: P 11.6: RC 10.8: D 4.*—June 1928 to March 1929 Engr.-Inspector, U. S. Bureau of Reclamation, Fairfield, Mont., on construction of Gibson Dam. *TT 0.8: P 0.8: RC 0.8.*—April 1929 to date Asst. Engr. (Civil), U. S. Engr. Office, Kansas City, Mo., Missouri River Improvement, inspection, training inspectors, reports, investigations and estimates. *TT 2.9: P 2.9: RC 2.5.*—*TT 19.7: P 19.7: RC 14.5: D 4.* Refers to C. A. Betts, M. Chase, F. J. Clark, A. H. Fuller, G. C. Haydon, H. Hodgman, M. K Snyder.

184

(16) ORCUTT, FORREST GLENN, 237 Custom House Bldg., Denver, Colo. (Age 34. Born Cedar Rapids, Iowa.) Registered Prof. Engr., State of Iowa.—1926 B. S. in Civ. and Irrigation Eng., Colo. State Agri. Coll. *TT 4: P. 4.*—Feb. to Sept. 1926 Recorder and Chf. of Party, Humphry Corporation, Houston, Tex., torsion balance survey, Geo.-Physical Dept. planetable work, mapping. *TT 0.4: SP 0.2: P 0.2: RC 0.2.*—Sept. 1926 to Aug. 1929 Instrumentman, Chf. of Party and Res. Engr., Iowa State Highway Comm., on surveys, construction, grading, paving, bridging, channel change (river). *TT 2.4: SP 0.5: P 1.9: RC 0.4.*—Aug. 1929 to June 1931 Asst. City and County Engr., City and County of Sheridan, Wyo., city paving, water-works, sewers, airport and golf course layouts, topographic mapping, county roads, bridges, surveys, irrigation systems and dams. *TT 1.9: P 1.9: RC 1: D 0.4.*—June 1931 to date Jun. Engr., Bureau of Public Roads, Denver, Colo., in charge of Employment Census, Dist. No. 3. *TT 0.4: SP 0.4.*—*TT 9.1: SP 1.1: P 8: RC 1.6: D 0.4.* Refers to T. W. Allen, E. C. Gwillim, C. E. Learned, L. M. Martin, A. E. Palen.

185

(12) OSBORNE, EDWARD GLENN, 4006 Burke Ave., Seattle, Wash. (Age 39. Born Omaha, Nebr.) 1915 B. S. C. E., Univ. of Wash. *TT 4: P 4.*—June 1910 to Sept. 1911 Chainman to Inspector, with New York Central Ry., on maintenance, and (after Feb. 1911) with Stone & Webster Eng. Corporation, on construction of Bellingham-Mt. Vernon Interurban Ry. in Washington. *TT 0.6: SP 0.6.*—June 1915 to Jan. 1917 Transitman, Alaska Gastineau Min. Co., Thane, Alaska, on construction of 10 000-ton crusher and retreatment plant, mine railroad, power plants, transmission lines, etc. *TT 0.8: SP 0.8.*—Feb. to May 1917 Computer on valuation, Southern California Edison Co., Los Angeles, Cal. *TT 0.2: SP 0.2.*—May to Aug. 1917 Draftsman, and June 1919 to Jan. 1920 Asst. County Engr. in charge of grading, paving and highway construction, with Gila County, Globe, Ariz. *TT 0.6: SP 0.1: P 0.5: RC 0.5.*—June 1918 to June 1919 Asst. City Engr. Globe, in charge of paving, water plant, etc. *TT 1: P 1: RC 1.*—Jan. to June 1920 Transitman, Miami (Ariz.) Copper Co., on construction of mill buildings, steel and concrete construction. *TT 0.5: P 0.5.*—July 1920 to June 1921 Transitman, and June 1921 to date Res. Engr., Washington State Dept. of Highways, on construction. *TT 11.6: P 11.6: RC 10.5.*—*TT 19.2: SP 1.6: P 17.6: RC 12.* Refers to T. G. McCrory, C. C. More, R. M. Murray, L. V. Murrow, O. A. Piper, R. H. Wilson.

186

(2) PANDYA, ANANT HIRALAL, 3 Hancock Park, Cambridge, Mass. (Age 22. Born Bhavnagar, India.) 1931 M. S., Mass. Inst. Tech. *TT 4: P 4.* Refers to H. K. Barrows, C. M. Spofford.

187

(13) PARK, KENNETH FOOTE, 2665 Third Ave., Sacramento, Cal. (Age 40. Born Mineral Park, Ariz.) 2 years student in Civ. Eng., International Correspondence Schools. ——1909 to 1916 in British Columbia, Ontario and Alaska, on mineral property surveys, Rodman and Chainman for various railroads, etc., and Govt. Office, Juneau, Alaska, etc. *TT 1.3: SP 1.3.*——1916 to 1924 (about 4 months each year, except 1 year during War) in private practice of surveying in Washington and British Columbia for L. R. McConnell, R. S. Ling, S. H. Baxter and others. *TT 3: P 3: RC 3.*——Feb. 1924 to April 1926 not on engineering, except a few weeks on surveys.——April 1926 to Sept. 1928 and Sept. 1929 to June 1930 with Fred H. Tibbetts, San Francisco, Cal., first as Transitman and Res. Engr. on location and construction surveys in Mountain, Foot-Hill and Valley Irrigation Dists., flumes, tunnels, siphons, irrigation and drainage, levee construction, and after Sept. 1929 Res. Engr. in charge of all field work for several reclamation and levee districts, right-of-way surveys, construction, small structures, etc. *TT 3.3: P 3.3: RC 2.3.*——Feb. to Sept. 1929 Inspector, Pacific Gas & Elec. Co., Bureau of Tests and Inspection, Emeryville, Cal., in charge of construction of gas holder (8 000 000 cu. ft.), Oakland, steel pipe fabrication, castings, electrical equipment. *TT 0.5: P 0.5: RC 0.5.*——June 1930 to Feb. 1931 Inspector, U. S. Engr. Office, Sacramento, in responsible charge of levee construction, small structures, diamond drill borings on proposed damsite. *TT 0.7: P 0.7: RC 0.7.*——March to July 1931 Night Supt., Le Tourneau Constr. Co., Stockton, Cal., in charge of construction, Boulder Dam to Boulder City highway. *TT 0.3: P 0.3: RC 0.3.*——Sept. 1931 to Jan. 1932 Engr. and Supt., Capital Constr. Co., Sacramento, in charge of highway construction and sections, cost keeping, reports. *TT 0.3: P 0.3: RC 0.1.*——Sept. 1928 to Feb. 1929, July to Sept. 1931 and Jan. 1932 to date Engr. and Supt. Pacific Coast Constr. Co., on bank-protection works, retards, current deflectors and river and property surveys. *TT 0.8: P 0.8: RC 0.8.*——*TT 10.2: SP 1.3: P 8.9: RC* 7.7. Refers to R. P. Bryan, R. Matthew, F. H. Tibbetts, R. G. Wadsworth, H. I. Wood.

188

(13) PARKER, THOMAS CLAIR, Zion National Park, Springdale, Utah. (Age 45. Born Queen City, Mo.) June to Nov. 1924 student, Highway Eng. course, Utah Agricultural Coll.——Jan. 1907 to Jan. 1914 Rodman, Chainman and Levelman, 2 years with Denver, Laramie & Northwestern R. R. Co., then with Laramie Poudre Irrigation Co., Ft. Collins, Colo. *TT 3.5: SP 3.5.*——Jan. 1914 to June 1917 Transitman and Chf. of Party, 2 years with Delta Land & Water Co., Milford, Utah, on surveys, construction, canals, roads, dams and subdivisions, then with Utah-Idaho Sugar Co., Shelley, Idaho, on location, construction, branch railroads and sugar plant. *TT 1.7: SP 1.7.*——June 1917 to Oct. 1919 Lieut. of Infantry, 21st Engrs., U. S. Army, on construction of railway bridges. *TT 2.3: P 2.3.*—— Oct. 1919 to May 1924 Res. Engr., Utah Highway Comm., Salt Lake City, Utah, on location, construction, design, plans and inspections. *TT 4.6: P 4.6: RC 2: D 1.1.*——Nov. 1924 to Jan. 1927 Field Engr., Utah Copper Co., Bingham, Utah, on location, construction of railroad bridges, in charge of field parties. *TT 2.1: P 2.1: RC 2.1.*——Feb. 1927 to Sept. 1928 Chf. Eng. Inspector Supt., U. S. Bureau of Public Roads, Juneau, Alaska, on location, designs, construction of highways, trails, inspections, reports and plans. *TT 1.6: P 1.6: RC 1: D 0.7.*——Sept. 1928 to June 1929 Inspector, California Highway Comm., San Rafael, Cal., on inspections, material tests and reports. *TT 0.7: P 0.7.*——June 1929 to date Associate Engr., National Park Service, San Francisco, Cal., being Park Engr. of Zion and Bryce National Parks, on estimates, reports and plans, under general direction of Chf. Engr., in charge of park engineering and construction. *TT 2.7: P 2.7: RC 2.7: D 1.*——*TT 19.2: SP 5.2: P 14: RC* 7.8: *D 2.8.*——Refers to E. S. Borgquist, J. H. Diehl, A. van V. Dunn, E. B. Feldman, F. A. Kittredge, E. A. Nickel, D. W. Ross.

189

(1) PEARSON, THEO PORTER, 2701 Woolworth Bldg, New York City. (Age 28. Born Orangeburg, S. C.) 1 year student, Univ. of S. C. *TT 0.5: P 0.5.*——June 1925 to March 1927 Chf., Survey Party, until April 1926 with Palm Beach Ocean Realty Co. and after May 1926 with Braxton Eng. Co. *TT 1.2: SP 0.4: P 0.8: RC 0.8*——March 1927 to date Contract Estimating Engr., Standard Dredging Co., estimating and assisting Mgr. *TT 5: P 5: RC 5.*——*TT 6.7: SP 0.4: P 6.3: RC 5.8.* Refers to A. A. Fries, E. M. Graves, W. F. Lineberger, G. W. Sackett, M. J. Young, G. A Youngberg.

190

(11) PEDIGO, JAMES ALAN, 1202 Hall of Records, Los Angeles, Cal. (Age 25. Born Hopkinsville, Ky.) 1924 to 1925 student, Vanderbilt Univ. *TT 0.5: P 0.5.*——May 1925 to April 1926 Draftsman, Tennessee State Dept. of Highways and Public Works, on plans, grade, estimate and drainage of highways. *TT 0.5: SP 0.5.*——April to Aug. 1926 Office

Engr. with R. E. Meade, Civ. Engr., Florence, Ala., planning subdivisions, preparing tract maps, street plans, estimates and surveys. *TT. 0.1: SP 0.1: D 0.1.*—Sept. 1926 to date with Los Angeles County Road Dept., until July 1929 as Jun. C. E. Draftsman on design of highways, streets, storm drains and highway structures, then Senior C. E. Draftsman, in charge of design of pedestrian tunnels, highway drainage structures and storm drains, and since Aug. 1930 Bridge Designer, Bridge Div., being responsible for design and architecture of grade separations, pumping plants and bridges. *TT 5: SP 0.3: P 4.7: RC 1.6: D 5.*—*TT 6.1: SP 0.9: P 5.2: RC 2.6: D 5.2.* Refers to W. D. Armstrong, R. H. Baker, W. H. Baker, Jr., E. A. Burt, O. F. Cooley, W. J. Fox, F. B. Gridley, R. A. Muenster.

191

(2) **REIGELUTH, RAY JOHN,** 58 Waverly St., New Haven, Conn. (Age 47. Born Waverly, N. Y.) 1908 B. A., and 1909 M. A., Yale Coll. *TT 2: P 2.*—Prior to 1924 (3 years) student, International Correspondence Schools.—1920 to 1926 with C. W. Blakeslee & Sons, until 1921 as Supt. on three road jobs in Branford, Conn., designed and developed batch trucking for concrete highways; 1921 to 1922 Asst. Supt. on Glastonbury Road, Hartford to New London, being in charge of 8 miles; built batch-measuring device of wood boxes for loading batch trucks; after 1922 Gen. Supt., about one year on macadam and concrete roads (12 miles) at New Milford, Conn., 2 years on Tomlinson Bridge, New Haven, over 9 months in charge of 650-ft. sea wall and dock for City of New Haven, 1 year on Freight St. Bridge, Waterbury and about 9 months on reconstruction of State High Bridge at Westbrook, Conn.; suggested some designs for spreading abutments by inverted concrete. *TT 7.4: P 7.4: RC 7.4: D 1.7.*—1927 to 1929 with Blakeslee-Rollins Corporation, being associated with J. W. Rollins in charge of two piers in Mid-Hudson Bridge, Poughkeepsie, N. Y., assisted in designing and directing methods for righting east caisson, including special design for clam shell bucket to work at unusual depth and a drag line and bucket for removing mud from under caisson. *TT 2.9: P 2.9: RC 2.9.*—1930 to date Gen. Supt. with C. W. Blakeslee & Sons, Inc., one year in charge of East St. sewage-disposal plant, designed plant layout and self-sustaining coffer dam (96 by 206 ft. with 19-ft head), in charge of sewage-disposal plant, Newington, Conn. (about 4 months) and of pumping station for city of Hartford (about 5 months); at present Vice-Pres. of C. W. Blakeslee & Sons, Inc., Treas., Dwight Bldg Co. and Connecticut Quarries Co., Pres., Certified Sand Co. (designed plant), and Asst. Treas., Blakeslee-Rollins Corporation. *TT 1.7: P 1.7: RC 1.7: D 0.3.*—*TT 14: P 14: RC 12: D 2.*—Refers to W. V. Barry, C. J. Bennett, C. Blakeslee, H. L. Blakeslee, H. R. Buck, C. A. Emerson, Jr., J. W. Rollins, Jr.

192

(7) **RUSKIN, SAMUEL JOHN,** 2988 Elmhurst, Detroit, Mich. (Age 21. Born Winnipeg, Man., Canada.) 1931 B. Sc., Univ. of Mich. *TT 4: P 4.*—March to Sept. 1928 Draftsman, until May 1928 with Ainsworth Mfg. Corporation, then Briggs Mfg. Corporation, both of Detroit, details and minor layouts of mechanical parts and (after May) automobile-body parts. *TT 0.3: SP 0.3.*—July 1929 to Feb. 1930 Draftsman, American Blower Corporation, Detroit, details and layouts of heating and ventilating apparatus. *TT 0.3: SP 0.3.*—Oct. 1931 to date Draftsman, Michigan State Highway Dept., details and computations. *TT 0.2: SP 0.2.*—*TT 4.8: SP 0.8: P 4.* Refers to L. M. Gram, N. Van Eenam, J. S. Worley.

193

(1) **SETHMANN, MARTIN WILLIAM,** 1717 Gates Ave., Brooklyn, N. Y. (Age 29. Born Brooklyn, N. Y.) 1928 B. S., N. Y. State Coll. of Forestry, Syracuse Univ. *TT 2: P 2.*—Sept. 1929 to date (nights) student, Brooklyn Polytechnic Inst.—Summers 1927 and 1928 Field Asst. on Pack Demonstration Forest, Warrensburg, N. Y., being Rodman, Chainman and Instrumentman with survey party, and (1928) on road location and construction, water supply layout and installation, building layout for new campsite, etc.—Dec. 1929 to Feb. 1930 with Pres. of Borough of Queens, N. Y., as Eng. Asst., Dept. of Sewers, on record maps and preliminary estimates for sewer construction. *TT 0.1: SP 0.1.*—Feb. 1930 to date Eng. Asst., Board of Transportation, New York City, on preparing and supervising field data for survey parties, preparing and studying field conditions for design and revision, preparing and computing estimates on subway construction. *TT 2: P 2.*—*TT 4.1: SP 0.1: P 4.* Refers to H. Fishgrund, C. M. Madden.

194

(6) **SETTLE, JOSEPH EDWARD,** 3 Manor Pl., Charleston, W. Va. (Age 44. Born Fayetteville, W. Va.) Registered Prof. Engr., State of Florida—1903 to 1907 (2½ years) student in engineering, West Virginia Univ. *TT 1: P 1.*—1904 to 1905 Rodman and

Instrumentman, Virginian R. R. *TT 0.5: SP 0.5.*—1907 to 1910 Instrumentman, in charge of party on surveys, Draftsman, Inspector and Res. Engr. on small water and sewer system installations, etc., under various engineers. *TT 3: P 3: RC 1.*—1910 to 1911 Instrumentman and Draftsman with B. M. Green, M. E., Charleston, W. Va., on surveys. *TT 1: P 1.*—1911 to 1914 Chf. Engr., Booner Coal & Coke Co., on surveys, maps, design and construction. *TT 3: P 3: RC 3.*—1914 to 1917 with his father and brother, Civ. and Min. Engrs., Fayetteville, W. Va., on surveys; also Office Assessor, Fayette County. *TT 1: P 1: RC 1.*—1917 to 1918 Res. Engr., Chicago, Wilmington & Franklin Coal Co., Virden, Ill. *TT 1.3: P 1.3: RC 1.3.*—1918 to 1920 Chf. Engr. and Asst. Gen. Supt., Continental Coal Corporation, Pineville, Ky., and Chf. Engr., Liberty Coal & Coke Co., Straight Creek, Ky.; 1918 to 1922 Owner and Operator of coal mines, Kentucky Straight Creek Coal Co., Gen. Mgr., H. L. Cory Coal Co., Pineville, Ky., and Cons. Engr., Pineville. *TT 2: P 2: RC 2.*—1922 to 1925 Cons. and Contr. Engr., Pineville, on design and supervision of sewers, Town of Pineville, Ky.; Res. Engr. on road construction, Bell County; and with Kentucky Utility Co., locating transmission lines, surveying steam plant site, sewers, roads, surveys and data, Cumberland River Investigations and reports on adjacent coal properties, etc. *TT 3: P 3: RC 3: D 1.*—1925 to 1927 Cons. Engr., Orlando Fla., subdivision layout, drainage construction, street paving, cemetery improvements and design. *TT 2: P 2: RC 2: D 2.*—1927 to date Cons. Engr., Charleston, W. Va., on work for various cities, towns, etc., including design and supervision of construction of storm and sanitary sewer systems, street paving, and water-supply and distribution systems, a 600-ft. concrete viaduct over a railroad, a rapid sand filtration plant, a continuous filtration swimming-pool with locker rooms, river development, etc. (given in detail in application). *TT 5: P 5: RC 5: D 5.*—*TT 22.8: SP 0.5: P 22.3: RC 18.3 D 8.*—Refers to L. H. Davis, W. S. Downs, C. P. Fortney, J. W. Hall, O. M. Jones, M. J. McChesney, E. H. Morris.

195

(16) SHANE, HENRY KAMSLER, 3214 Bellefontaine, Kansas City, Mo. (Age 29. Born Kansas City, Mo.) 1923 B. S. in E. M., Univ. of Kans. *TT 4: P 4.*—1923 to 1927 2d Lieut., Corps of Engrs., U. S. Army, at Ft. Sam Houston, Tex., first year with Second Engrs., and remainder of time with Office of Engr., 8th Corps Area, in charge of survey parties on topographic mapping; completed Leon Springs Military Reservation and Ft. Bliss and Vicinity Fire Control Maps, San Lorenzo Creek Quadrangle, etc. *TT 3.8: P 3.8: RC 3.*—1927 to 1928 Res. and Field Engr., Bexar County Highway Dept., San Antonio, Tex.; on improvements from preliminary surveys, to final construction, including concrete bridges and culverts, widening and straightening roads, including macadam, rock asphalt and concrete dips and curbs. *TT 0.9: P 0.9: RC 0.9.*—1928 to 1929 Res. Engr. with Black & Veatch, Cons. Engrs., Kansas City, Mo., 4 months on highway materials and bridge inspection, Jackson County, had charge of distribution pipe-line extensions, Ada, Okla., pipe-line improvements, Evansville, Ind., about 11 miles of cast-iron pipe and distribution improvements; also on construction of 20 000 000-gal. covered concrete reservoir, made surveys for new booster pump station and two 10 000-gal. steel standpipes. *TT 1.8: P 1.8: RC 1.2.*—1929 to 1930 Cons. Engr. at San Antonio, Tex., made several minor subdivisions and some plats and geologic studies. *TT 0.9: P 0.9: RC 0.9.*—1931 to date Jun. Civ. Engr., Inspector and Chf. Inspector, under Dist. Engr., on Missouri River Improvements, at present Jun. Engr., Hydr. Div., Kansas City, on stream flows and flood control. *TT 0.6: SP 0.2: P 0.4: RC 0.2.*—*TT 12: SP 0.2: P 11.8: RC 6.2.* Refers to E. B. Black, J. F. Brown, A. C. Bux, E. H. Coe, P. A. Diehl, D. H. McCoskey, C. D. Whitmire.

196

(1) SLOANE, HAROLD ANTHONY, 271 Madison Ave., New York City. (Age 36. Born Brooklyn, N. Y.) C. E., International Correspondence Schools.—June 1914 to April 1915 Draftsman, Field Inspector, Rodman and Chainman with Clarke, Howe & Homer, Archts., Providence, R. I. *TT 0.4: SP 0.4.*—April 1915 to Sept. 1917 with J. W. Bishop Constr. Co., New Bedford, Mass., as Asst. Supt. and later Supt. in charge of construction of industrial, textile and power-plant work. *TT 2: SP 0.5: P 1.5: RC 1.5.*—Sept. 1917 to March 1918 Gen. Supt., American International Ship Bldg. Corporation, Hog Island, Philadelphia, in charge of construction of ten concrete shipways (each 900 ft. long), four 1200-ft. fitting-out basins, of dredging, cofferdams, etc. *TT 0.5: P 0.5: RC 0.5.*—March to Dec. 1918 Res. Engr., Dupont Eng. Co., Bldrs., in charge of construction of Old Hickory (Nashville, Tenn.) Powder Plant, etc. *TT 0.8: P 0.8: RC 0.8.*—Dec. 1918 to April 1922 Mgr. of Constr., James Alexander Constr. Co., Memphis, Tenn., in charge of construction and field work on industrial, public and semi-public buildings, levees, railroads, bridges, slaughter-

houses, etc., in the south, middle and southwestern states. *TT 3.3: P 3.3: RC 3.3.*—April 1922 to Jan. 1923 Gen. Supt., Casper-Ranger Constr. Co. and Milford (Mass.) Constr. Co., in charge of erection of 1 400 houses, including construction of roads, streets, sanitary and storm sewers, disposal plant, electric lighting, foundations and superstructures of buildings; designed and erected village of Medford Hills, Milford. *TT 0.7: P 0.7: RC 0.7: D 0.7.*—Jan. to June 1923 Supt., H. A. Harris, Inc., New York City, in charge of construction on Public School 95, Jamaica, N. Y., laid out building, made alterations, installed foundations, erected steel and brick work. *TT 0.5: P 0.5: RC 0.5.*—June 1923 to May 1928 Pres., H. A. Sloane's Contrs. Service; originated quantity survey estimates for use of general contractors (average building construction over $200 000 000 annually). *TT 4.8: P 4.8: RC 4.8.*—May 1928 to May 1929 Asst. Chf. Engr., Wm. Kennedy Constr. Co., Brooklyn, N. Y., in charge of all estimating, construction, analyzing material requirements, inspecting manufacture and design of plant equipment; construction included Williamsburg Savings Bank, Brooklyn, N. Y. *TT 1.1: P 1.1: RC 1.1: D 1.1.*—May 1929 to June 1930 Chf. Engr., S. S. Roth & Bros., Inc., New York City, in charge of all field construction, purchases, estimating, plant layout and organization; firm's work included construction of two large New York State projects and rebuilding of runways, hangars and general construction of Roosevelt Field (Airport), Mineola, N. Y. *TT 1.1: P 1.1: RC 1.1: D 1.1.*—June 1930 to date Pres., H. A. Sloane, Inc., and H. A. Sloane Estimating Service, Inc., Estimators and Cons. Engrs., furnishing estimates and design of plant layouts and of heavy engineering work. *TT 1.7: P 1.7: RC 1.7: D 1.7.*—*TT 16.9: SP 0.9: P 16: RC 16: D 4.6.* Refers to C. S. Boardman, L. L. Brown, J. Clausnitzer, H. J. Deutschbein, A. Verduin.

197

(1) SPRINGER, REID CRAIGG, P. O. Box 122, Woodside, N. Y. (Age 42. Born Muscatine, Iowa.) Licensed Surveyor, State of California.—1909–1910 student, Univ. of Minnesota. *TT 0.5: P 0.5.*—Jan. 1913 to Jan. 1915 Chainman and Surveyor, Richmond Cedar Works, Norfolk, Va., surveys and railroad construction. *TT 1.7: SP 0.2: P 1.5: RC 1.5.*—Jan. 1915 to Dec. 1918 Conducting Officer (Transport), British Remount Comm. —Feb. to Sept. 1919 Chainman with Bennett & Duling, Los Angeles, Cal., on surveys. *TT 0.2: SP 0.2.*—Oct. 1919 to May 1920 Chainman and Instrumentman with K. V. Morin, Los Angeles. *TT 0.3: SP 0.3.*—June 1920 to Jan. 1921 Time-Check Paymaster, Auditing Dept., Eng. Office, Southern California Edison Co., Big Creek Project, Los Angeles.—Feb. to Dec. 1921 Asst. to Supt. Bent Bros., Los Angeles, constructing San Dimas Canyon Dam, Los Angeles County Flood Control. *TT 0.4: SP 0.4.*—Jan. 1922 to Jan. 1923 Asst. Engr., Seaboard Eng. Co., Los Angeles, on city subdivision and municipal improvements. *TT 1: P 1: RC 1.*—Jan. to July 1923 Asst. Engr. with Leeds & Barnard, Los Angeles, on marine surveys, New Port Beach, Cal. *TT 0.5: P 0.5.*—Aug. 1923 to Feb. 1924 Surveyor, Union Oil Co., Los Angeles, on surveys and construction for oil development. *TT 0.2: SP 0.2.*—Feb. 1924 to Feb. 1925 with Bureau of Water, on primary triangulation survey, Colorado River Aqueduct. *TT 1: P 1.*—April 1925 to Jan. 1926 and Nov. 1926 to April 1927 in private practice as Surveyor, map making, etc. *TT 1.2: P 1.2: RC 1.2.*—Jan. to Nov. 1926 Asst. Engr., Dept. of City Transit, Philadelphia, on construction and final estimate, Broad St. Subway. *TT 0.8: P 0.8.*—April to Sept. 1927 with W. I. Browne, City Surveyor, Elmhurst, N. Y. *TT 0.4: P 0.4: RC 0.4.*—Sept. 1927 to Nov. 1928 Res. Engr., City Housing Corporation, New York City, on municipal improvements, topographical and landscape, design and construction, Munsey Park, Manhasset, N. Y. *TT 1.2: P 1.2: RC 1.2: D 1.2.*—Jan. 1929 to Dec. 1930 Transitman, Long Island R. R., on surveys and estimates, construction, line and grade, etc., for grade-crossing elimination. *TT 1.5: SP 0.5: P 1.—TT 10.9: SP 1.8: P 9.1: RC 5.3: D 1.2.* Refers to G. N. Adams, P. H. Albright, T. J. Carlile, R. Eberlin, N. M. Imbertson, R. L. Patterson.

198

(13) STOKES, CHARLES ERIC, 1749 L St., Fresno, Cal. (Age 21. Born Oakland, Cal.) 1931 B. S., Coll. of Eng., Univ. of Cal. *TT 4: P 4.* Refers to R. E. Davis, C. Derleth, Jr., B. A. Etcheverry, F. S. Foote

199

(15) STRIEBECK, WILLIAM HERBERT, Jr., George West, Tex. (Age 36. Born Breslau, Tex.) 1914–1916 student, Texas Agricultural & Mechanical Coll. *TT 1: P 1.*—July 1916 to March 1917 Chainman, Rodman, Instrumentman and Computor on railroad valuation work. *TT 0.6: SP 0.1: P 0.5.*—Sept. 1917 to May 1918 Instrumentman, July to Oct. 1919 and May 1921 to May 1924 Asst. County Engr., Wichita County Highway, on location, grading and drainage structures, construction of concrete, asphalt and gravel roads. *TT 3.9:*

P 3.9: RC 2: D 2.—May 1918 to June 1919 Draftsman and Observer, Intelligence Sec., U. S. Army, 90th Div. Headquarters, St. Mihiel and Meuse-Argonne Fronts, Army of Occupation, Germany, working out situation maps. *TT 0.9: SP 0.2: P 0.7: RC 0.5: D 0.5.*—Oct. 1919 to March 1920 Instrumentman and Acting Asst. Engr., Fort Worth & Denver City Ry., on valuation. *TT 0.4: P 0.4: RC 0.2.*—Sept. 1920 to March 1921 Asst. Engr., Atascosa County Highways, grading and drainage structures. *TT 0.5: P 0.5: RC 0.5.*—May 1924 to May 1925 Asst. Engr., Clay County Highways, on locations, plans, grading and drainage structures. *TT 1: P 1: RC 0.7: D 0.7.* March 1925 to July 1928 State Licensed Surveyor, County Surveyor of Live Oak County. *TT 3.2: P 3.2: RC 3.2.*—July 1928 to date Res. Engr., State Highway Dept. of Texas. *TT 3.7: P 3.7: RC 3.7: D 3.7.*—*TT 15.3: SP 0.3: P 15: RC 10.8: D 6.9.* Refers to T. W. Bailey, G. Gilchrist, T. E. Huffman, T. J. Kelly, J. B. Nabers, R. J. Potts, W. H. Prentice, Jr.

200

(2) TAYLOR, DAVID YEATON, 22 Usher Road, West Medford, Mass. (Age 23. Born West Medford, Mass.) 1931 B. S. in C. E., Tufts Coll. *TT 4: P 4.*—Aug. 1931 to date Transitman, City of Medford, Mass., on surveys, calculations, mapping, and design of minor structures. *TT 0.3: SP 0.3.*—*TT 4.3: SP 0.3: P 4.* Refers to H. P. Burden, R. W. Lefavour, F. N. Weaver, E. H. Wright.

201

(7) TOMEK, ARTHUR OTTO, 1519 Nineteenth St., Two Rivers, Wis. (Age 36. Born Two Creeks, Wis.) 1918 B. S. in C. E., Marquette Univ. *TT 4: P 4.*—May to Dec. 1918 with Engr. Corps, U. S. Army, Camp Lee, Va. *TT 0.3: SP 0.3.*—1919 to 1920 Draftsman, Wisconsin State Highway Comm. *TT 0.5: SP 0.5*—1920 to 1922 Asst. City Engr., Manitowoc, Wis., on plans and specifications of concrete streets, sewers, etc., being in charge of Drafting Dept. *TT 1.6: SP 0.4: P 1.2: RC 1: D 0.2.*—1922 to 1923 drafting, design of steel and wood furniture, Hamilton Mfg. Co., Two Rivers, Wis. *TT 0.5: SP 0.5.*—1923 to 1925 with Detroit, Toledo & Ironton R.R. (Ford Motors, Dearborn, Mich.) as Asst. Chf. of Party on railroad survey and construction, also on subdivision work, being responsible for instrument and topographical work. *TT 1.5: SP 0.5: P 1: RC 1.*—1925 to Jan. 1928 with F. F. Mengel, Wisconsin Rapids, Wis., as Instrumentman, Chf. of Party and Res. Engr. on survey, plans and supervision of construction of concrete highways and bridges. *TT 2.6: SP 0.4: P 2.2: RC 1.2: D 1.*—April 1928 to 1931 County Highway Engr., Manitowoc County, in charge of construction, surveys and records. *TT 2.3: SP 0.3: P 2: RC 1.5: D. 0.5.*—1931 to date City Engr., Two Rivers, Wis., supervising construction and design of all city projects. *TT 0.9: P 0.9: RC 0.5: D 0.4.*—*TT 14.3: SP 3: P 11.3: RC 5.2: D 2.1.* Refers to W. C. Buetow, J. Donohue, G. E. Hill, W. H. Lange, F. F. Mengel, O. C. Rollman.

202

(7) WELTNER, WILLIAM, 12450 Barlow Ave., Detroit, Mich. (Age 40. Born Peoria, Ill.) Registered Civ. Engr., State of Michigan. 1916 B. C. E., Univ. of Mich. *TT 4: P 4.*—July 1916 to April 1917 Engr. with George B. Pyke, Traverse City, Mich., made topographic survey and maps of state hospital grounds (880 acres) and buildings. *TT 0.7: P 0.7: RC 0.7: D 0.4.*—April to Nov. 1917 Engr. Inspector, Milwaukee Sewerage Comm., gave line and grade and inspected tunnel and open cut sewers, etc. *TT 0.3: SP 0.3.*—Nov. 1917 to July 1918 Draftsman, Wisconsin State Highway Comm., Milwaukee, on road work. *TT 0.3: SP 0.3: D 0.3.*—July 1918 to May 1920 with Michigan State Highway Comm., first as Asphalt Inspector, then Draftsman on road work, and after May 1919 Res. Engr. on asphalt roads. *TT 1.4: SP 0.4: P 1: RC 1: D 0.3.*—May 1920 to April 1927 with C. W. Hubbell and Martin R. Fisher, until July 1920 as Draftsman on sewer design, then Chf. of party on sewer construction, paving, surveys, etc., from July to Oct. 1920 being rated as Instrumentman. *TT 6.7: SP 0.2: P 6.5: RC 6.5: D 0.2.*—April 1927 to April 1928 Village Mgr. and Village Clerk, Plymouth, Mich., in charge of all departments, etc. *TT 1: P 1: RC 1: D 0.2.*—April 1928 to date Chf. of Field Party, City Engr.'s Office, Detroit, in charge of sewer construction and survey work. *TT 3.9: P 3.9: RC 3.9.*—*TT 18.3: SP 1.2: P 17.1: RC 13.1: D 1.5.* Refers to M. R. Fisher, J. R. Hendry, C. W. Hubbell, L. G Lenhardt, M. F. Wagnitz.

203

(13) WHEATON, HERBERT HUDSON, 2336 Channing Way, Berkeley, Cal. (Age 33. Born Easton, Minn.) 1922 B.S. in Civ. Eng., Univ. of Wis.—1929 M.S. in Civ. Eng., Univ. of Cal. *TT 4: P 4.*—Sept. 1921 to Aug. 1922 (while student) Instructor in Topographical Eng., and at Summer Surveying Camp, Univ. of Wisconsin. Sept. 1922 to June 1923 Instructor in Eng., Sept. 1924 to Sept. 1926 Asst. Prof., and Sept. 1926

to June 1928 and June 1929 to Aug. 1931 Associate Prof., of Eng., Fresno (Cal.) State Coll., also Director, Summer Surveying Camp, and (Oct. 1925 to May 1930) Engr. in charge of design, construction and maintenance of stadium for Fresno State Coll. Association. *TT 6.4: P 6.4: RC 1: D 0.3.*—Aug. 1928 to May 1929 and Aug. 1931 to date graduate student in Civil Engineering, Univ. of California. *TT 10.4: P 10.4: RC 1: D 0.3.*—Refers to P. F. Brown, R. E. Davis, B. A. Etcheverry, S. T. Harding, J. E. Mackie, M. P. O'Brien, F. C. Scobey.

204

(11) WILSON, ROBIN STUART, 1021 West Thirty-second St., Los Angeles, Cal. (Age 32. Born Altenschlirf, Germany.) 1932 B. S. in Civ. Eng., Univ. of So. Cal. *TT 4: P 4.*—Sept. 1921 to Aug. 1922 Operating Foreman, Burnham Chemical Co., Westend, Cal., constructing and operating pilot plant in borax and potash development, electric installation, etc. *TT 0.5: SP 0.5.*—Dec. 1923 to Jan. 1927 Safety Engr., Nevada Consolidated Copper Co., Ray, Ariz., in charge of accident and fire prevention, ventilation studies, first aid and mine rescue training, equipment and operations. *TT 2.9: SP 0.3: P 2.6: RC 2.6.*—Jan. to Oct. 1927 Supt., Sierra Salt Co., Keeler, Cal., in charge of gathering fields and salt shipping, controling water on field, etc. *TT 0.8: P 0.8: RC 0.8.*—Nov. 1927 to May 1929 Furnace Foreman, Mercury Mining Syndicate, McDermitt, Nev., in charge of operation, maintenance, repairs of 100-ton rotary kiln furnace and condenser plant, etc. *TT 1.3: SP 0.2: P 1.1.: RC 1.1.*—June 1929 to Sept. 1930 Surveyor, Yellow Pine Co., gold mining, Stibnite, Idaho, laying out claims, pipe and power lines, etc., surface and underground surveying, drifts, shafts, connections, etc., earth-dam design, mapping, timbering, logging, steam plant operation, etc. *TT 1.3: P 1.3: RC 0.6.*—Feb. 1931 to Jan. 1932 (while student) Asst. in Surveying, Univ. of Southern California.—*TT 10.8: SP 1: P 9.8: RC 5.1.* Refers to J. C. Clausen, R. M. Fox, W. H. Leffingwell, A. M. Strong, D. M. Wilson.

205

(12) WONG, QUEN YUEN, 4303 Eighth Ave. N. E., Seattle, Wash. (Age 24. Born Canton, China.) 1931 B. S. in C. E., Univ. of Wash. *TT 4: P 4.* Refers to G. E. Hawthorn, C. C. More, R. G. Tyler.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

206

(11) BRUNER, EDWIN LEWIS, Assoc. M., 307 Hill St., Los Angeles, Cal. (Elected Aug. 9, 1920.) (Age 43. Born Los Angeles, Cal.) Professional Certified Archt. and Registered Civ. and Structural Engr., State of California.—June 1909 to Aug. 1910 Draftsman, Los Angeles & Salt Lake R. R. *TT 0.6: SP 0.6.*—Sept. 1910 to April 1913 with Los Angeles Aqueduct until May 1911 as Field Draftsman on locating party, then Transitman (Chf. of Party) on surveys, and after Oct. 1912 in charge of surveys for underground distributing system for Power Bureau. *TT 2.1: SP 0.3: P 1.8: RC 1.8.*—May 1913 to Feb. 1917 Designer, City Engr.'s Office, Los Angeles, until April 1916 on large storm sewer systems, including preliminary work, drainage, hydraulic and structural design; Squad Leader, supervising work, then on miscellaneous structures, pavements, right-of-way computations and descriptions, Vrooman Act work, responsible for designs for important street improvements. *TT 3.6: SP 0.3: P 3.3: RC 1: D 3.3.*—March to Aug. 1917 Draftsman, and later Civ. Engr., California State Highway Comm., making location maps, right-of-way descriptions, estimates, paper location and survey computations for mountain highways. *TT 0.5: P 0.5.*—Aug. 1917 to March 1920 Civ. Engr. and Designer, California State Harbor Comm., on design and inspection of various harbor improvements, also Squad Leader in charge of design on reinforced concrete and timber bulkhead wharves, piers and pier sheds. *TT 2.1: SP 0.6: P 1.5: RC 1.5: D 1.5.*—March 1920 to Oct. 1921 Structural Engr. and Bldg. Designer, Spreckles Sugar Co., San Francisco, Cal., in charge of design on various buildings and engineering structures. *TT 1.7: P 1.7: RC 1.7: D 1.*—Oct. 1921 to May 1922 Engr. and Designer, Bridge Dept., California Highway Comm., Sacramento, on design of highway bridges, culverts, etc. *TT 0.6: P 0.6: RC 0.3: D 0.6.*—May 1922 to Oct. 1923 Structural Engr. and Archt., with Trewhitt Shields Co., Fresno, Cal., being Designer, Estimator and Inspector on buildings, including schools, factories, warehouses, packing plants, ice houses, stores and garages, structural frame in reinforced concrete, steel and heavy timber. *TT 1.4: P 1.4: RC 1.4: D 1.*—Oct. 1923 to date Structural Engr. and Archt. until Dec., 1930 for Hamm, Grant and Bruner, Inc., then for Grant & Bruner, Ltd., Los Angeles, Cal., in charge of structural and architectural design and estimates for bidding on office buildings, factories, stores, cold-storage plants, laundries, warehouses, garages, packing

plants, etc. (over $8 000 000), and since Jan. 1930 in charge of structural and architectural design, estimates and specifications, for reinforced concrete, structural steel and heavy timber buildings, since 1925 being Vice-Pres., and after Jan. 1930 also Treas. of company. *TT 8.4: P 8.4: RC 8.4: D 6.2.*—*TT 21: SP 1.8: P 19.2: RC 16.1: D 13.6.* Refers to D. M. Baker, R. R. Fisher, K. C. Grant, N. F. Jahn, H. Z. Osborne, C. L. Peck.

207

(7) CORSON, HAROLD HAWLEY, Assoc. M., 979 Chesterfield Rd., Birmingham, Mich. (Elected Dec. 3, 1928.) (Age 45. Born Birmingham, Mich.) 1907 B. S. in C. E., Univ. of Mich. *TT 4: P 4.*—June 1907 to June 1910 with Detroit River Tunnel Co., as Inspector (1 year), Laboratory Asst. on physical testing (5 months), Chf. Inspector (10 months) and Asst. Div. Engr. (10 months), on construction. *TT 2.4: SP 0.6: P 1.8: RC 0.8.*—March to July 1911 Instrumentman, U. S. Reclamation Service. *TT 0.3: P 0.3.*—July to Nov. 1911 Party Chf. for E. B. Henry, Klamath Falls, Ore., on surveys and plans. *TT 0.4: P 0.4: RC 0.4: D 0.1.*—May to Dec. 1912 Office Engr., California-Oregon Power Co., Yreka, Cal., on design. *TT 0.6: P 0.6: RC 0.6: D 0.6.*—Feb. to Oct. 1913 Chainman and Levelman, California Highway Comm. *TT 0.3: SP 0.3.*—Feb. to Aug. 1914 and May 1922 to Aug. 1923 in private practice on construction. *TT 1.7: P 1.7: RC 1.7: D 0.4.*—Aug. 1914 to March 1915 Draftsman, Ford Motor Co., Detroit, on building design. *TT 0.6: P 0.6.*—March to Sept. 1915 and March 1916 to Jan. 1917 Engr. of Standards, Wilson Body Co., Detroit. *TT 1.3: P 1.3: RC 1.3: D 0.5.*—Sept. 1915 to March 1916, Jan. to Aug. 1917 and March to Nov. 1919 Engr. of Accounting Dept., Detroit Trust Co., cost accounting. *TT 1.8: P 1.8: RC 1.8.*—Aug. to Dec. 1917 Locating Engr., Detroit Constr. Co., Pontiac, Mich., on location. *TT 0.3: P 0.3: RC 0.3.*—Dec. 1917 to Oct. 1918 1st Lieut., and Oct. 1918 to March 1919 Capt., 30th Engrs., U. S. Army, field work in United States and France. *TT 1.3: P 1.3: RC 1.3.*—Nov. 1919 to May 1922 Engr. of Standards, Towson Body Co., Detroit. *TT 2.5: P 2.5: RC 2.5: D 1.*—Aug. to Oct. 1923 Party Chf. on Location, Oct. 1923 to May 1924 Office Engr. on design, and May 1924 to Dec. 1927 Field Engr. on design and construction, Michigan State Highway Dept. *TT 4.3: P 4.3: RC 4.3: D 2.1.*—Dec. 1927 to date Village Engr., Birmingham, Mich. *TT 4.2: P 4.2: RC 4.2: D 2.*—*TT 26: SP 0.9: P 25.1: RC 19.2: D 6.7.* Refers to G. C. Dillman, W. C. Hirn, C. A. Melick, M. Osgood, E. D. Rich, W. W. Tefft, J. H. Wasson.

208

(1) HAW, ELMER PERKINS, Assoc. M., Box 292, Ancon, Canal Zone (Elected April 7, 1915.) (Age 44. Born Maiden Rock, Wis.) 1910 B. S. in C. E., Iowa State Coll. *TT 4: P 4.*—July 1910 to May 1912 Rodman and Levelman, and Nov. 1912 to Feb. 1914 Transitman, Isthmian Canal Comm., acting as Instrumentman and Draftsman on surveys and engineering work for Panama Canal. *TT 1.5: SP 1.5.*—May to Nov. 1912 Res. Engr., R. W. Hebard & Co., Contrs., on construction of street railway system for City of Panama. *TT 0.5: P 0.5: RC 0.5: D 0.2.*—Feb. 1914 to April 1915 Dist. Engr., Dept. of Public Works, Govt. of Panama, being Asst., in charge of office designs, estimates and field construction of public works, including buildings, bridges, wharves and municipal improvements. *TT 1.2: P 1.2: RC 0.7: D 0.4.*—April 1915 to April 1926 with Panama Elec. Co., Panama, operating street railways, until 1917 as Engr. in charge of maintenance-of-way and structures, then Gen. Supt. in responsible charge of planning and executing program of all maintenance and improvements and reconstruction of physical plant, including power plant, car barns and track and roadway. *TT 4.5: P 4.5: RC 4.5: D 0.5*—April 1926 to Oct. 1929 member of firm, Wright, Haw & Jean Guardia, Lt., Archts., Engrs. & Contrs., Panama, being directly in charge of construction contracts (approx. $500 000 annually), principally residences, apartment houses and clubs, also on architectural and engineering work involving plans, estimates and specifications for various construction projects (over $1 000 000 annually). *TT 3.5: P 3.5: RC 3.5: D 0.5.*—Oct. 1929 to July 1930 Engr., Grebien & Martinz, Inc., Panama, Bldg. Contrs., in charge of estimates and construction. *TT 0.7: P 0.7: RC 0.7.*—July 1930 to date with Dredging Div., Panama Canal, first six months not on engineering work; since Jan. 1931 in charge of design and construction of two reinforced concrete spillways. *TT 1: P 1: RC 1: D 0.2.*—*TT 16.9: SP 1.5: P 15.4: RC 10.9: D 1.8.* Refers to G. C. Bunker, J. G. Claybourn, T. Guardia, R. W. Hebard, R. C. Jones, E. W. Wilson, A. S. Zinn.

209

(16) HILTON, JOSEPH KENNICOTT, Assoc. M., 709 First Ave., Clarion, Iowa. (Elected May 28, 1923.) (Age 45. Born Mt. Savage, Md.) Student, Univ. of Iowa and Northwestern Univ.; completed 1 year in Liberal Arts and 3 years in Eng. *TT 1.5: P 1.5.*—Summer 1909 Draftsman for C. C. Stowell, County Engr., Rockford, Ill., detail drawing

on concrete factory construction, inspecting and testing materials.—June 1911 to Jan. 1912 with Dept. of Public Works, Evanston, Ill., inspecting paving and testing materials. *TT 0.3: SP 0.3.*—July 1912 to March 1914 with Chicago & Northwestern Ry., as Rodman (4 months), Draftsman in Chf. Engr.'s Office (4 months), and Bridge Inspector on construction in Iowa (1 year). *TT 0.8: SP 0.8.*—March 1914 to July 1915 and March 1916 to March 1919 Asst. County Engr., until May 1915 with Crawford County and remainder of time with Sac County, Iowa, on road and bridge design and superintendence of construction. *TT 4.5: P 4.5: RC 4.5: D 4.5.*—July 1915 to March 1916 on surveys and plans for drainage districts, Greene County, Iowa. *TT 0.7: P 0.7: RC 0.7: D 0.7.*—March 1919 to March 1927 as County Engr., Wright County, Iowa, designed and superintended construction of 150-mile County Road System, and as Res. Engr., Iowa State Highway Comm., superintended the construction of 60 miles of state work. *TT 8: P 8: RC 8: D 8.*—March 1927 to date Div. Maintenance Engr., Iowa State Highway Comm. *TT 5: P 5: RC 5: D 5.*—*TT 20.8: SP 1.1: P 19.7: RC 18.2: D 18.2.* Refers to C. C. Coykendall, C. H. Currie, W. D. Maxwell, R. E. Robertson, R. R. Zack.

210

(16) RAEDER, ELMER WARREN, Assoc. M., 1020 Thirteenth St., Boulder, Colo. (Elected Assoc. M. July 16, 1928.) (Age 40. Born St. Louis, Mo.) 1916 B. S. in Civ. Eng., Univ. of Cal. 1927 M. S., Univ. of Colo. *TT 4: P 4.*—Nov. 1916 to Sept. 1917 Draftsman detailing steel bridges and buildings, until Feb. 1917 with Stupp Bros. Bridge & Iron Co., St. Louis, Mo., then with American Bridge Co., Gary, Ind. *TT 0.9: P 0.9.*—Sept. 1917 to Feb. 1919 2d Lieut., Air Service, U. S. Army; after Dec. 1918 being Commanding Officer, 52d Balloon Co. *TT 1.3: P 1.3: RC 0.2.*—Feb. 1919 to Jan. 1920 Asst. Engr., Dept. of Bridges and Bldgs., St. Louis, detailing and designing concrete buildings and bridges. *TT 0.8: P 0.8.*—Jan. 1920 to Oct. 1923 Estimator and Designer, Stupp Bros. Bridge & Iron Co., detailing, designing and contracting for steel mill buildings, etc. *TT 3.8: P 3.8: RC 3: D 1.*—Oct. 1923 to Sept. 1925 ill.—Sept. 1925 to date with Univ. of Colorado, until July 1926 as graduate student, and Engr., Constr. Dept., and since Sept. 1926 Instructor (2 years) and Asst. Prof., Dept. of Civ. Eng.; summers with Western Pipe & Steel Co., San Francisco (1927) and State Highway Dept., Denver, Colo. (1928 and 1930). *TT 5.5: P 5.5: RC 3.5: D 0.5.*—*TT 16.4: P 16.4: RC 6.7: D 1.6:* Refers to S. W. Bowen, W. E. Brockway, L. F. Copeland, C. L. Eckel, N. W. Funk, H. J. Gilkey, W. C. Huntington, O. T. Reedy.

211

(5) ROYER, ROBERT STUART, Assoc. M., 207 Builders Exchange Bldg., Richmond, Va. (Elected Feb. 6, 1912.) (Age 47. Born Roanoke, Va.) 1905 B. S., and 1906 C. E., Va. Pol. Inst. *TT 4: P 4.*—July 1905 to Jan. 1906 Rodman, Topographer, Levelman and Masonry Inspector, Norfolk & Western Ry. Co. *TT 0.3: SP 0.3.*—Jan. 1906 to July 1908 Instrumentman, and Asst. Engr. on track laying and yard construction, Virginian Ry. *TT 2: P 2: RC 1.*—July 1908 to Aug. 1912 member of firm, McGhee & Royer, Cons. Engrs., Roanoke, Va., in charge of construction of Mill Mt. incline, preliminary and location, Roanoke Mt. Airy Ry. and extension, Virginia, Carolina Ry., Engr. in charge of design and construction of suburban developments, etc. *TT 4: P 4: RC 4: D 1.*—Aug. 1912 to Aug. 1918 City Mgr., Fredericksburg, Va., in complete charge of all city operations, including water and gas works, reorganization of departments, design and building of surface treated gravel streets. *TT 6: P 6: RC 6: D 2.*—Aug. 1918 to Jan. 1919 with U. S. Army, as 1st Lieut., Engrs., in training at Camp Humphreys.—Jan. to Nov. 1919 Business Mgr., Chamber of Commerce, Alexandria, Va., making industrial survey and promoting City Mgr. form of government. *TT 0.9: P 0.9.*—Nov. 1919 to Jan. 1921 Gen. Mgr., with Rogers & Royer, Contrs., on construction of concrete state highways and bridges, including 6 miles of concrete road. *TT 1.1: P 1.1: RC 1.1.*—Jan. 1921 to June 1923 Pres., Royer-Ferguson Co., Inc., on construction of concrete highways and bridges, including about 25½ miles of concrete roads in Maryland, Virginia and North Carolina, also on experimental work on design and building of unloading plants, concrete roads, etc. *TT 2.5: P 2.5: RC 2.5.*—June 1923 to Jan. 1928 member of firm, Baker-Royer Co., in charge of engineering and construction on 41 miles of concrete highway for State of Virginia, sewerage system, sewage-disposal and filtration plants and water mains for Warrenton, Va., a railroad underpass, sewerage system and Imhoff tank for Delmar, Del., etc. *TT 4.5: P 4.5: RC 4.5.*—Jan. 1928 to date Pres., R. Stuart Royer, Inc., engineering and construction on waterworks and sewers; constructed 7 miles of 6-in. gas line, constructed 20-in. water-supply line for and was appointed Superv. Engr. for Henrico San. Dist. No. 1. *TT 4.2: P 4.2: RC 4.2.*—*TT 29.5: SP 0.3: P 29.2: RC 23.3: D 3.* Refers to A. H. Bell, G. M. Bowers, T. Eliett, E. M. Hastings, L. O. Hines, G. H. Massey, C. L. Reid, C. C. Robinson, A. M. Traugott.

FROM THE GRADE OF JUNIOR

212

(1) CLOGSTON, PERLEY WALTER, Jun., 565 Academy St., New York City. (Elected July 14, 1930.) (Age 32. Born Williamstown, Vt.) 1922 B. S., Dartmouth Coll. 1923 C. E., Thayer School of Civ. Eng. *TT 4: P 4.*—April to Sept. 1922 Rodman, Delaware & Hudson R. R.—April 1923 to Oct. 1925 Draftsman and Estimator, Illinois State Highway Dept., plans, estimates and proposals for grading, paving and bridges. *TT 1.8: SP 0.8: P 1.*—Oct. 1925 to March 1930 Transitman and Chf. of Survey Party, Kenn-Well Contr. Co., on construction of a power plant, substations and office buildings, inspecting pile driving, etc. *TT 3.3: SP 1.1: P 2.2: RC 0.2.*—March 1930 to date Chf. of Survey Party with Marc Eidlitz & Son, at New York Hospital, laying out piers on basement floors of New York Stock Exchange for future construction, New Eye Inst., for Presbyterian Hospital. *TT 1.9: P 1.9: RC 1.9.*—*TT 11: SP 1.9: P 9.1: RC 2.1.* Refers to R. R. Benedict, J. L. Bogart, F. E. Cudworth, P. S. Dow, R. Fletcher, R. R. Marsden, A. P. Richmond, Jr.

213

(9) DORITY, ALEX OUSLEY, Jun., 1020 Temple Bar Bldg., Cincinnati, Ohio. (Elected Feb. 24, 1931.) (Age 30. Born Pembroke, Ky.) Sept. 1920 to May 1923 student, Vanderbilt Univ. *TT 1.5: P 1.5.*—July 1923 to April 1925 with Algernon Blair Constr. Co., Montgomery, Ala., estimating and Asst. Supt. placing reinforced concrete on building construction, also field cost control. *TT 1.3: SP 0.4: P 0.9.*—May to Dec. 1925 with Florida Radio Telegraph Co., Miami, Fla., on design and construction of small communications system. *TT 0.6: P 0.6: RC 0.6: D 0.6.*—Jan. 1926 to July 1927 with Seaboard All Florida Ry., Miami and West Palm Beach, Fla., as Transitman, and Foundation Inspector on bridges. *TT 1.2: SP 0.4: P 0.8.*—July 1927 to Feb. 1928 Field Party Chf., City of Ft. Lauderdale, Fla., on rivers and harbors development and city improvement projects, general layout and inspection of rolling bascule bridges, reinforced concrete sea walls, and part of new city water-supply system, also establishing permanent leveling system throughout city. *TT 0.6: P 0.6: RC 0.6.*—March to Aug. 1928 with A. B. Davidson, Gen. Contr., Athens, Ohio, estimating and supervising construction, reinforced concrete highway bridges and buildings. *TT 0.5: P 0.5: RC 0.5.*—Aug. 1928 to Jan. 1929 Concrete Inspector, Corps of Engrs., U. S. Army, Louisville Dist., Ohio River Dam No. 51, Golconda, Ill., in charge of concrete field control and test specimens, aggregates, cylinders, cement, etc. *TT 0.5: P 0.5.*—Feb. 1929 to date with Cincinnati (Ohio) Union Terminal Co., as Draftsman on general design, estimating and plans for construction of new freight and passenger railroad terminal, Office Engr., Office of Engr. of Constr., recording and preparing charts of grading settlement, checking plans, charting and recording construction schedules and progress, etc. *TT 3.1: P 3.1: RC 1: D 1.*—*TT 9.3: SP 0.8: P 8.5: RC 2.7: D 1.6.* Refers to R. J. Bear, J. M. Belknap, A. M. Bouillon, J. C. Bussey, W. McL. Hall, G. A. Pierce, R. Ritter, G. P. Stowitts, O. Wells.

214

(7) DUVALL, ARNDT JOHN, Jun., 250 South Syndicate Ave., St. Paul, Minn. (Elected March 15, 1926.) (Age 28. Born Stillwater, Minn.) 1925 B. S. in Civ. Eng., Univ. of Minn. *TT 4: P 4.*—July 1925 to March 1926 Designer and Draftsman, Bridge Dept., Missouri State Highway Dept., designing, drafting and checking plans for highway bridges. *TT 0.7: P 0.7.*—March 1926 to date Engr. with Toltz, King & Day, Inc., Engrs. and Archts., St. Paul, Minn., until April 1926 as Draftsman on plans for power plant, April to Oct. 1926 Inspector on concrete warehouse building, Oct. 1926 to Nov. 1927 estimating, designing and drafting plans for buildings, Nov. 1927 to March 1928 Engr.'s Supt. on power plant addition for City of Austin, Minn., and since March 1928 designing, estimating, making appraisals, writing specifications and drafting plans for buildings, sewers, etc., and supervising construction of several structures. *TT 6: P 6: RC 2.3.*—*TT 10.7: P 10.7: RC 2.3.* Refers to F. Bass, A. S. Cutler, M. W. Hewett, H. M. Hill, W. E. King, J. I. Parcel.

215

(11) LAMB, GILES HARVEY, Jr., Jun., 510 North McClelland St., Santa Maria, Cal. (Elected March 12, 1923.) (Age 32. Born Yates Center, Kans.) Licensed Civ. Engr., State of California. 1922 B. S. in Civ. Eng., Univ. of Kans. *TT 4: P 4.*—Sept. 1918 to Jan. 1919 Private, U. S. Army, Student Army Training Corps, Univ. of Kansas.—June 1922 to July 1923 Asst. County Engr., Douglas County, Kans., inspecting 100-ft. truss bridge with concrete floor, computing stresses, designing and making plans and estimates for pony trusses and reinforced concrete structures, reports, surveys, etc., superintending road maintenance. *TT 0.8: SP 0.2: P 0.6: D 0.6.*—July 1923 to June 1925 County Engr.,

Chase County, Kans., in charge of county road and bridge work, computing stress, designing, making plans and estimates, inspecting construction of reinforced concrete structures up to 50-ft. spans, superintending construction and maintenance of earth roads; Executive for Board of County Commrs. *TT 1.9: P 1.9: RC 1.9: D 0.8.*—June 1925 to Sept. 1926 with Black & Veatch, Cons. Engrs., Kansas City, Mo., on municipal improvements, 2 months as Asst. Resident, and 3 months Res. Engr., on pavements, and 10 months Res. Engr. on sanitary sewers. *TT 1.2: SP 0.1: P 1.1: RC 0.4.*—Sept. 1926 to Jan. 1927 associated with S. G. Chamberlin, Licensed Surveyor, San Fernando, Cal., subdivision maps, Torrens title work, establishing lost lines and corners, street grades, curb and walk stakes. *TT 0.3: P 0.3: RC 0.3.*—Jan. 1927 to March 1930 preparing estimates and bids from plans, Timekeeper, Purchasing Agt., etc., at Los Angeles, Cal., until Feb. 1928 with Bebek & Brkich, Gen. Sewer Contrs., then with Boske & Konjevod, Paving Contrs., and after March 1929 in private practice with L. B. Konjevod, Paving Contr. *TT 2.3: SP 0.7: P 1.6: RC 1.6.*—March to June 1930 Topographical Draftsman, Bureau of Water and Power, Los Angeles, calculating capacity and embankment yardage of earth reservoirs, and co-ordinates of traverse. *TT 0.2: SP 0.2.*—June 1930 to date Asst. Res. Engr., Grade III, California Highway Comm., Dists. III and V, inspecting reinforced-concrete structures, designing and controlling Portland cement concrete mixes, field and office work, etc. *TT 1.6: P 1.6.*—*TT 12.3: SP 1.2: P 11.1: RC 4.2: D 1.4.* Refers to J. C. Clausen, P. A. Diehl, W. C. McNown, A. J. Smith, N. T. Veatch, Jr.

216

(15) **MESSER, MERTON WILLIAM,** Jun., 1804 Tulip St., Baton Rouge, La. (Elected Feb. 24, 1931.) (Age 29. Born Concord, N. H.) Feb. 1923 to May 1924 Draftsman and Rodman, New Hampshire State Highway Dept., Concord, N. H., on roads and culverts. *TT 0.7: SP 0.4: RC 0.3.*—May 1924 to Oct. 1926 Draftsman, Alabama State Highway Dept., Montgomery, Ala., detailing, tracing, estimating and checking concrete decks, I-beam spans, trusses, abutments, bents, footings, culverts, etc. *TT 2.4: P 2.4.*—Oct. 1926 to Oct. 1927 Bridge Detailer, Tennessee Dept. of Highways and Public Works, Nashville, Tenn., on similar work, also designing. *TT 1: P 1: D 0.3.*—Oct. 1927 to Sept. 1929 Bridge Detailer, Alabama State Highway Dept. and Alabama Bridge Corporation, Montgomery, general office work on 15 steel toll bridges, detailing, designing and estimating railroad overhead and underpass work for grade separation, checking steel schedules, shop plans, preliminary layouts from field survey notes, government permit maps concerning War Dept. requirements, laying grades, estimating earthwork, designing concrete structures, preparing final estimates, etc., detailing concrete through rainbow arch bridge (7 spans, 60 to 135 ft.) and instructing Res. Engr. and Contr.; also on building design and estimating for contractors, etc. *TT 1.9: P 1.9: D 0.5.*—Sept. 1929 to Feb. 1931 Designer-Draftsman, Texas & Pacific Ry. Co., Dallas, Tex., on layout, investigation and design for large terminal, and grade separation program for City of Ft. Worth, Tex., designed viaducts, subways, under and over passes, bridges, etc.; designed, detailed and traced timber falsework, concrete deck girders, piers, footings, abutments, retaining walls, etc., also layout work on reconditioning large bridges, etc. *TT 1.4: P 1.4: RC 1.4: D 1.*—Feb. 1931 to date Bridge Designer, Louisiana State Highway Comm., Baton Rouge, La., on bridge design, also some work on continuous spans, etc. *TT 1: P 1: RC 1: D 0.4.*—*TT 8.4: SP 0.4: P 8: RC 2.4: D 2.2.* Refers to R. P. Boyd, J. C. Early, F. E. Everett, P. Moller, H. E. Tobey.

217

(14) **RIDDELL, JOHN CALVIN,** Jun., 1014 Rector Bldg., Little Rock, Ark. (Elected Oct. 21, 1924.) (Age, 32. Born Enterprise, Kans.) 1924 B. S. in Civ. Eng., Kansas State Coll. *TT 4: P 4.*—Sept. 1921 to Feb. 1922, summers 1922 and 1923 and June 1924 to date Engr. with Gordon Walker, Gen. Contr., Little Rock, Ark., on various buildings in Kansas, Nebraska, Missouri and Arkansas, including hospitals, schools, office, store and bank buildings, court house ($45 000 to $1 250 000), a garage ($50 000), dairy plant ($15 000), masonic temple ($225 000), jail ($160 000), dam ($12 000), remodeling a theatre ($30 000) (total over $4 400 000); since Jan. 1928 also made estimates on 123 buildings ($8 000 to $1 900 000). *TT 10.6: SP 0.8: P 9.8: RC 2.6: D 0.6.*—*TT 14.6: SP 0.8: P 13.8: RC 2.6: D 0.6.* Refers to L. E. Conrad, W. D. Dickinson, A. C. Galt, L. J. N. Keliher, W. W. Means, E. U. Stevenson.

218

(1) **SCHOEN, HARRY,** Jun., 30 Olive Pl., Forest Hills Gardens, N. Y. (Elected June 6, 1927.) (Age 27. Born New York City.) 1927 C. E., Cornell Univ. *TT 4: P 4.*—Summer 1922 and 1925 with Eugene Schoen, Archt., New York, as Plan Boy, tracing, etc., and Outside Representative, supervising work, preparing plot plans, details, cost reports, etc.—

Summers 1923, 1924 and 1926 with Leddy & Moore, Inc., Mason Bldrs., New York City, as Labor Foreman, Engr.'s Helper, Asst. Job Engr., Job Engr., Asst. to Supt., etc., checking sub-contractor's work, ordering materials, in charge of progress reports, adjustments, etc.—March to June 1927 with Spencer, White & Prentis, Inc., New York City, as Asst. Job Engr. on Fox Theatre, Brooklyn, N. Y., excavation and foundations, had charge of reports, worked on lines and levels, supervised concrete pouring, being in charge of night shift on completion of job, also Supt. of Constr., in charge of finishing foundation work on Lincoln Park Bldg. and Griffith's Piano Bldg., Newark, N. J. *TT 0.3: P 0.3: RC 0.1.*—June 1927 to April 1928 and Sept. 1928 to Jan. 1929 with Leddy & Moore, Inc., first as Supt. of Constr. in charge of excavation, rock-testing, drilling, blasting, concreting and reinforcing on footing, foundations, pits, subsurface walls, buttresses and retaining walls, supervised steel erection and brick work, had charge of lines and levels, inspected sub-contractor's work (except mechanical equipment), etc., on St. Regis Hotel alterations and addition ($5 500 000), New York City, acted as Asst. Supt. (under Gen. Supt.) on remainder of job; after Sept. 1928 Asst. Supt. and Job Engr. on Bedell Bldg., New York City, also laid out foundation work and supervised pile driving on new refinery buildings for Nichols Copper Co., Laurel Hill, N. Y. *TT 1.2: P 1.2: RC 1.2: D 0.2.*—Feb. 1927, April to Sept. 1928 and Feb. 1929 to date with Eugene Schoen, Inc., first supervising contractor's work, details, tracings, structural design, etc.; April to Sept. 1928 Archt.'s Outside Representative in charge of construction, efficiency layouts for industrial offices, some designing, structural design, interior decorating on bank and department store, etc.; since Feb. 1929 on similar work, including charge of sub-contractors work on Bronx Branch, Public National Bank & Trust Co., let contracts and supervised work, approved requisitions, etc., on several bank alteration jobs, handled interiors on apartments, etc., had charge of Macy and Stern Bros. Dept. Stores interior alterations and structural changes; supervised structural and decorative work and furnishings on S.S. Leviathan Night Club ($100 000), in charge in Hoboken and Boston; Office Mgr., some structural design, handled orders, contractors, etc., some sales work, etc. *TT 3.5: P 3.5: RC 3.5: D 3.2.*—*TT 9: P 9: RC 4.8: D 3.4.* Refers to H. C. Balcom, F. A. Barnes, J. E. Perry, G. E. J. Pistor, F. J. Seery, T. W. White.

The Board of Direction will consider the applications in this list not less than thirty days after the date of its issue.

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ANNUAL CONVENTION

YELLOWSTONE NATIONAL PARK

July 6, 7, 8, and 9, 1932

The Reading Room of the Society is open from 9:00 A. M. to 5:00 P. M. every day, except Sundays and holidays; from May to September, inclusive, it is closed on Saturdays at 12:00 M.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is a file of 274 current periodicals, the latest technical books, and the room is well supplied with writing tables.